THE REGULATION OF RIVERS

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INTRODUCTION

1. The Improvement of Waterways is a Commercial Necessity.—Waterways have always held a prominent part in the commercial life of the world. Rivers and the seas constitute nature's especial contribution to the development of material progress through their advantage as fundamental routes of communication.

Economic transportation demands the removal of impediments to navigation. Harbors must be given an easy approach and ample accommodations for ocean shipping. Canals form essential links of waterways where natural barriers obstruct their continuity on great trade routes. Rivers require the deepening of their channels at shoal places and other essential improvements to fit them in full measure for their service to commerce.

2. Both Theory and Experience Essential in Regulation of Rivers.—The regulation of rivers involves the control of flowing water. It therefore is based upon hydraulic laws. The bed and banks of a stream are unstable in greater or less degree. These modifications are produced by the river currents, but the principles involved are as yet but imperfectly understood. The purpose is to so control the flow that a favorable channel shall form in places where it has been inadequate.

Regulation is not wholly a question of the application of known hydraulic formulæ. Many works of improvement have been disappointing when designed with too great a reliance upon the exactitude of values involving variable factors, or with too serene a confidence that the effect will be favorable in all essential details if only the theoretical values are attained. Neither is the regulation of rivers entirely a proposition of adopting such design as has proved successful elsewhere. Many projects of this character have fallen short of the expected efficiency because of failure to adequately modify the design in adapting it to differences in regimen. The exponent of the finality of hydraulic theory proclaims for its results an adequacy which often proves fictitious; while the partisan of the sufficiency of practical experience assumes for it a potency which is essentially fortuitous. In no kind of endeavor is it more necessary to blend theory and practice so as to extend the scope of formulated

laws to include all the special uncertainties of fluvial development which "good practice" has so often been expected to insure.

3. Outline of the Treatment of the Subject.—It therefore seems important to give fundamental consideration to principles underlying operations; and this purpose has dominated the writing of this book. Not only are those scientific laws presented which are of proved relevance, but also other propositions which give promise of reducing many uncertainties of procedure to definite details of control; especial care has also been taken to discuss quite fully the methods and results of experimental investigations and other endeavors to secure a more complete knowledge of the laws involved. It is not as an art, but as an applied science that the regulation of rivers must especially develop; and in the measure that scientific principles govern will these public works prove successful.

A prior knowledge of such essential subjects as Surveying and Hydraulics is assumed, to the extent to which they form a part of the standard college course in civil engineering. Consequently the scope of the discussion of such introductory features includes only those topics which the author's experience has found necessary in order to insure an adequate comprehension of the whole subject. Figures, statistics, plans and operations are not given exhaustively; but they are, instead, summarized or selected as representative types to illustrate and elucidate fundamental principles. This ordered concentration of treatment, although more difficult to effect, should offer a better grasp and a truer perspective of the general subject than can any other arrangement. The many references in text and footnote are given not only to cite authority and to convey due acknowledgment, but also to particularly direct the attention to a paper or book of great value for desirable collateral reading on the subject under consideration. The complete reports of the Chief of Engineers, U. S. A., naturally constitute the origin of the greater part of the available information upon the general subject of the regulation of rivers in this country.

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CHAPTER I

COMMERCIAL CONSIDERATIONS

1. Engineering Works must be Justified by Their Service.— Engineering works are justified, not only as they exemplify sound theory and good practice in their construction, but also in the measure in which they accomplish the service for which they are planned. Not only must the design be true in theory and detail and be practicable in its application, the materials be adequate to their duty, and the workmanship be honest and thorough; but the service rendered by these works must be of such character that burdens will be reduced, conveniences increased, health and well-being conserved, or some other human advantage be secured by the undertaking to an extent which is at least commensurate with the cost. A twenty-story building is a reproach unless the income from it is sufficient to pay all charges. A city's sewer system provides no direct return; but its defense is the convenience of those served by it, and the safeguarding of the health of the community.

The more usual criterion of this justification is financial in character because the ordinary engineering venture is a commercial one. This is particularly true of works of a private character in distinction from public works, as the former are always projected for the purpose of producing revenue while the latter often are not justified by adequacy of direct earning power. In fact, most duties of a government are such that direct compensation for its activities is either partial or else is wanting; but in every case the sum of indirect or collateral advantages must bring the total benefit to at least an equivalent of the outlay in order to justify such governmental action.

An excellent example of the private undertaking, which must be financially remunerative, is a railway; which meets all current expenses of operation and maintenance, taxes, interest, salaries and similar expenses, and also pays fair dividends to its stockholders. This does not signify that each mile of railway produces a revenue proportionate to its cost; if this were so, few large bridges could be built, as the proportionate revenue from freight and passenger

service over them would rarely meet their capital and maintenance charges. Nor does it mean that all its property is directly revenue-producing at all; the passenger and freight stations merely permit and facilitate the collection and distribution of the commodities which produce revenue, which is based primarily upon transportation mileage. The economic necessity of the railway system is that those station facilities, which directly produce no revenue, shall indirectly aid and encourage traffic to an extent commensurate with their cost; and that the bridge and terminal improvements (so enormously expensive when considered on the mileage basis) shall adequately economize and facilitate the general handling of the railway's business; so that the income from traffic moved shall pay all expenses and charges resulting from the operation of the properties as a whole.

Illustrating public works whose justification is indirect, are the improved highways of civilized nations which produce, in this country, no direct revenue at all; but whose value is measured by such collateral advantages as reduced expense of traffic over the improved road, increase of taxation values, non-interruption of teaming and driving, the saving of time, the facilitating of business and social intercourse, and other advantages affecting the prosperity and convenience of those within the influence of good roads. In this case the returns from the engineering work are in the form of benefits which have not the definiteness that inheres in the case of the undertaking whose criterion is revenue. To correctly apprehend the scope and intangible value of such indirect (but equally legitimate) advantages requires a more profound insight than does the commercial sense to foresee financial profits only; but the principle is plain and the expenditure is, as before, only justified when the aggregate balance of the advantages that will follow the inauguration of the project is in favor of the undertaking.

5. The Development of Highways, Waterways and Railways.—Railways, highways, waterways and the seas have constituted the particular agencies of social and commercial intercourse whose influence upon civilization has been incalculably great, and whose service in promoting the progress and prosperity of the world is very direct and impelling. All the civilized nations of antiquity and of the middle ages established extensive routes of trade and travel by roads, rivers or the sea; in fact, the advancement of communities and nations in prosperity has corresponded very closely to their development of such routes. Gradually the leading nations improved trans-

portation facilities until, a century ago, ocean commerce had become a leading factor in the world's activities; and highways (those of particular importance being scientifically constructed) were constituted the great arteries of interior communication. Bulletin 25, U. S. Office of Public Road Inquiries, says of the National Pike (which was built from Baltimore, over the Alleghanies through Wheeling to "the waters of the Ohio, traversing seven different states of the Union and covering 800 miles . . . . .) as many as twenty four-horse coaches could be counted in line at one time, and large broad-wheeled wagons covered with white canvas and carrying often 10 tons of merchandise drawn by six horses of superb form and strength could be seen at all hours of the day and at all points of the road. . . . . . It was indeed one vast and continuous caravan." Yet so great was the toll of time, labor and expense of hauling heavy and bulky freight over the roads, that the utilization of rivers was being rapidly extended by their improvement and by the construction of connecting canals in order to economize and facilitate commercial activities by the aid of boats. In fact, early in the nineteenth century all the progressive nations were becoming rapidly committed to a most extensive development of inland waterways, when the locomotive was made a practicable servitor; there no longer existed the vital necessity for the extensive improvement of rivers and construction of canals, and railways soon became (and have continued during the past century to be) the most potent of the three great agencies of interior transportation. This fact is indicated by the statement in S. Document 83, 59th Congress, 1st Session, that "since 1820 about fifteen billion dollars of property and money have been invested and accumulated in railroads within the United States." There exist, however, fundamental differences in the principle of their operation. Railways, from the nature of the case, must control both the roadbed and the transportation over it; while in the case of highways and waterways, transportation over them is open on equal terms (usually free) to individual teams or boats, making the logical and usual ownership of such routes a governmental function.

The U. S. Office of Public Roads estimates the cost (1911) of the 190,679 miles of improved roads of the country as amounting to $553,662,806. The total federal appropriation for improving our rivers from the origin of the government to Jan. 1, 1911 has been $346,812,362. Many advocates of the railway interests refer to the fact that the United States has expended more than six hundred

million dollars through its river and harbor appropriations as an unwarranted bonus to waterway interests. This charge overlooks three important considerations; the navigable rivers of the country are controlled by the government, and the cost of any works of improvement is consequently a necessary charge upon the government; about two-thirds as much of the "rivers and harbors" appropriations has been expended on the improvement of our harbors as upon that of the rivers and this benefits the railways fully as much as waterways; and railways have also benefited much from governmental aid in the form of gifts of money and bonds by municipalities, counties, and states, guaranties of bonds and interest and land grants and tariff remissions by the federal government, etc. Cleveland and Powell's "Railroad Promotion and Capitalization" gives a partial list of thirteen states, four counties and six cities whose subsidies to railways have amounted to nearly $150,000,000; and as this list is not complete for even the thirteen states, it is evident that the total financial aid from the whole country has been a large amount. The largest indirect bonus to railways has been the federal land grants, amounting to 158,286,627 acres, of which title to 108,153,252 acres has passed to the railways. The value to assign to this domain, larger than California or than all that part of the United States east of Ohio and north of Maryland, is uncertain; but computed at the price put upon government lands it would amount to more than $135,000,000. Evidently governmental aid to railways has at least equalled the amount expended upon interior waterways.

Although the unit cost of hauling freight over highways is relatively great, being from about 10 to 30 cents per ton-mile, and perhaps averaging 15 cents per ton-mile in this country, yet they necessarily constitute the usual medium for the general collection and distribution of commodities between transportation centers and individual patrons not immediately accessible to railways or waterways. The advantageous correlation of waterway and railway transportation is, however, not so evident; yet certain experiences in Europe and in this country indicate that a similar assembling and conveying of merchandise between the waterway and all its tributary territory by the railway is a frequent economic advantage. The remarkable development of our railways and the diminishing commerce of many of our waterways have led many to attribute to the former not merely a deserved preëminence, but virtual dominance of the field of inland commerce.

To understand the ascendency which the railways of the United

States sustain over the commercial use of inland waterways, it is necessary not only to bear in mind the inherent advantage of the railways in being able to reach any locality whatever in the continental domain; but one must also appreciate the main influences affecting the history of development of these two agencies. Previous to 1830, the only alternative to the teaming over highways was the use of interior waterways; and both roads and rivers and canals were developed and improved with increasing diligence by the various governmental authorities. The next thirty years witnessed the introduction and the gradual extension and development of the railways under private control, possessing the very definite advantages of adjustability of location and facilities to the traffic needs, reliability of service which escapes interruption because of ice or low water, and adaptability to the varied requirements of commerce. As the railway service thus entered its particular and broad field of commercial activity, the necessity of the extensive improvement of highways and waterways became less urgent; and yet, through the middle of the last century, the rivers and canals of the country sustained a creditable share in the transportation facilities of the republic.

6. Factors that Favored Rapid Railway Expansion in the United States.—During the last half of the nineteenth century, and particularly the sixth and seventh decades, there occurred a combination of conditions which tremendously accentuated the increasing preeminence of the railways, and gradually reduced many of the waterways to a position of minor importance.

The first of these considerations is the fact that the railways of this country were beginning to definitely realize the importance of their advantages over waterways in ability to extend their lines in the direction of the demands of traffic and to reach any district which is of commercial importance, instead of being confined to the river channels or to particular lines of topographical availability for canal construction; in opportunity to operate continuously throughout the year, without the interruption caused by the freezing over of the waterway in severe climates or by low waters leaving insufficient channel depths; in a lessening of distance, even when the routes are parallel, to six-tenths, seven-tenths or eight-tenths the distance by water; in greater safety of operation; in a saving of time due to a schedule speed several times that which is practicable on waterways; and in eliminating, for most localities, that portion of the cost of transportation involved in the transfer of commodities from car to

boat and from water to rail which is usually necessary in securing the advantage of the lesser cost of transportation by water.

The second consideration of significance in the cause of the relatively lessening importance of the waterways was the gradual consolidation of connecting railway lines into single systems. This began noticeably in the sixth decade, and has continued ever since. For example, the seven different companies which, in 1850, controlled the railways between Albany and Buffalo were united into one system in 1851. The administrative and economic advantages of such mergers became more and more apparent and influential in producing conditions which greatly economize and facilitate railway transportation. The aggregation of interests and capital thus occurring has given fullest opportunity for increasing enormously the efficiency of various details of such service.

The third serious influence of a half-century ago was the civil war, with the financial stringencies of that period. The scope of military operations practically effected the destruction of waterway traffic on more than half of the navigable channels of the country, which therefore went to the railways; and the business panics found the railway interests in far better condition to resist their adverse influence because of their relative compactness of organization and large capital. The task of recuperating from both of these very serious adversities of that time was a most difficult one for the waterway interests because of the relative impotence of the comparatively small capital of the numerous navigation companies acting independently of each other, as contrasted with the power of defense and growth inherent in the comparatively wealthy competing railway companies.

The fourth subject of especial consideration in this connection is the fact that gifts to the railways at this critical period by various federal, state, county and municipal governments aggregated many times the amount granted to waterways, although the improvement of rivers is fundamentally a governmental function in caring for its own property, while the building of railways is a private enterprise which controls its own capital. Complete figures are lacking, but the truth of the statement is evident from the fact that, while the total appropriations by the federal government for the improvement of rivers from its organization to the year 1871 was less than $21,000,000, the various governmental units of only four of the states (Massachusetts, Pennsylvania, Kentucky and Iowa) had donated to the railways subsidies amounting to more than

$33,000,000. The significance of this general fact is not so much in the relative amounts apparently given as aid, as in the fact that, in the case of the rivers it constituted the sole opportunity for making the channels more reliable, while in the case of the railways it was a bonus for increasing the effectiveness of private property built with invested capital and thus especially aided them at a time when the rival waterway interests were particularly vulnerable.

The fifth fact of particular moment at this critical period involved the question of discriminating railway tariffs. In order to secure the business, railway systems which competed with waterways inaugurated and developed the practice "freely to adjust their rates so as to meet water competition, and even destroy it."[1] Railways reduced rates on portions of their lines affected by the waterways, recouping themselves from the higher rates on other portions of their systems. "This has been the case with most of the great inland waterways, excepting the Great Lakes where the conditions of water and traffic approach those of open seas."[2] One instance of such discrimination, which occurred about sixty years ago, concerned river traffic from Macon, Georgia, over the Ocmulgee and Altamaha Rivers and the inland (coast) route to Savannah, carrying (with other bulky, imperishable freight) cotton bales at $1 each; the railroads met this rate and their speedier and more convenient service put the boats out of business. Then the railway rates were raised to $1.50 per bale; as a consequence the boats were again started, taking shipments at the former rate; but they were again forced to withdraw by a repetition of the same tactics of temporary railway tariff reduction. The Preliminary Report of the Inland Waterways Commission (1908) states on page 319 that "in many cases the railways are probably carrying goods at less than cost (at least if the traffic be charged with its proportion of fixed charges), for the purpose of shutting out water competition." A reduction of rates to a figure which does not include a just proportion of capital charges and maintenance, as well as of operation, is of course economically indefensible.

This combination of conditions in the third quarter of the last century, and existing in a country of comparatively small commerce at the time, gave to the railway interests an ascendency in the field of transportation in the United States that the succeeding years have

[1] President Delano of the Wabash Railroad, in *Railway Age Gazette*, January 6, 1911.

[2] Senate Document 325, 60th Congress, 1st Session, 1908, p. 12.

but intensified. The accumulated momentum of successful rivalry has served to increase their competitive resources in various essential particulars.

The discriminating rates, which were inaugurated to secure to each railway system all the business possible, crystallized (in the seventies) in a number of traffic associations covering different parts of the country and organized for the purpose of fixing and controlling railway rates. In the general region north of the Potomac and Ohio Rivers and east of the Mississippi River, railway rates were established as a percentage, in proportion to distance, of the base-rate plus fixed or terminal charges. Thus, taking the base-rate, New York to Chicago, as 25 cents per hundred pounds, and deducting 6 cents terminal charges, left 19 cents as the base-rate for mileage; for Indianapolis, distant 833 miles from New York City, the computation gave $\frac{833}{920} \times 19$, or $17\frac{1}{8}$ cents; to which the terminal charge of six cents was added, making the total Indianapolis rate $23\frac{1}{8}$ cents. South of the Ohio River the "basing point" system of tariffs prevailed, in which rates to trade centers having waterway or railway competition form the basis of the various schedules, to which competitive rate was added the local rate from such competitive point to any non-competitive place even though the local place were between the shipping station and the basing point. A similarly complex system of fixing rates as affected by competitive conditions at commanding trade centers on the basis of fixed differentials, and modified by various special considerations, controlled the railway tariffs from the Mississippi River to the Rocky Mountains; *e.g.*, the rate on grain from a town in Nebraska to the east consisted of the local rate to each locally commanding trade center plus the rate from each of those commanding centers to the east so adjusted that the total rate should be substantially equal by whatever route shipment was made. On transcontinental tariffs competitive conditions again controlled, this time due to ocean competition, which resulted in common rates on west-bound traffic from points east of the Missouri River to the Pacific coast, to which were added arbitrary charges to points at a distance from the coast; and the graded zone tariffs on east-bound freight. An example of such irrational methods is furnished by the statement that rates from St. Louis to some cities in Nevada have equalled that from New York to San Francisco, plus a rate from San Francisco to the Nevada localities which has in some instances exceeded the transcontinental portion of the total.

7. Resulting Conditions which have made Railways Preëminent. —In all these typical railway tariff systems, water and rail competition had a very great part in determining the basis of rates in all parts of the country; and, of course, this influence still persists. Objectionable features also were evident, the most serious of which was the frequent fact that rates from one competitive point to another were often less than that to intermediate places. For instance:

"The rates from Washington, D. C., to Mobile are less than those to Charlotte, N. C., less than half the distance; the all-rail rate on freight of the first class from New York to Brunswick, Ga., is 87 cents, the coastwise water rate being 57 cents, while the corresponding all-rail rate to Columbia and Augusta is $1.08. Most points half-way, or more than half-way, from Cincinnati to Mobile are charged higher rates than Mobile . . . in most instances the west-bound rates to intermediate towns several hundred miles east of the Pacific coast are higher than the through rates for the longer haul . . . the rates from St. Louis to Vicksburg for most classes and commodities are the same as to New Orleans; while the charges to Jackson in the same latitude as Vicksburg, but not on the river, are higher than to Vicksburg or to New Orleans . . . rates on imports and exports are made much lower than on like domestic goods when the carriers consider lower rates to be necessary to secure or to develop the traffic due to external trade."[1] "As the railroads will carry almost all articles of freight from New Orleans 396 miles to Memphis at the same rate as 214 miles to Natchez, shippers by water . . . have nothing to gain to points above Natchez. In the same way shippers have little to gain by using water transportation from St. Louis to points below Memphis, and nothing to points below Greenville. Consequently, packets plying between Memphis and Natchez must depend for their support on local business within the limits, so long as the railroads maintain these rates and can take care of the business."[2]

The frequent practice of the railways to charge more for a shorter than for a longer haul was so repugnant to the public sense of justice that the interstate commerce act of 1887 prohibited such charges "under substantially similar circumstances." This gave relief in a majority of cases, but did not alter the situation when the rate is made greater for a shorter haul, due to the influence of waterway competition; as the Interstate Commerce Commission and the United States Supreme Court have, in substance,

[1] Reprinted from Johnson and Huebner's "Railroad Traffic and Rates," Vol. 1., Copyright, 1911, by D. Appleton & Company.

[2] House of Representatives, Document 50, 61st Congress, 1st Session.

ruled that water competition does not constitute "substantially similar circumstances" in the meaning of that law. The Mann-Elkins act of 1910 amended the act of 1887 in a way to give further relief, allowing the Commission a wide discretion in this question and will apparently tend to prevent the railways from meeting water competition by the methods of past times by such provisions as prohibiting a railway from subsequently raising a rate which has been lowered to meet waterway competition.

The second great advantage which has gradually assumed definiteness under the splendid growth of railway business is the development of extensive terminals (including land, buildings and advantageous freight-handling equipment) to facilitate and economize the necessary loading and unloading of commodities. A considerable portion of railway capital is invested in this essential portion of the railway's property; and when we note that expert estimates put the terminal costs on freight as averaging from 24 percent (as in the rate computations of the Trunk Line and Central Traffic Associations already referred to) to 40 percent of the total cost of a thousand-mile haul,[1] even when such facilities are extensively developed, we realize the particular advantage which adequate terminals secure.

The third consideration of notable superiority involves the development, to a high degree of perfection, of various commercial conveniences and needs which are both attractive and valuable to the shipper. The railways are financially responsible for freight entrusted to their care; their business is so vast that they have established agencies in commercial centers where shippers conveniently secure information and assistance in arranging for the desired service; and their universal custom of issuing through bills of lading that are negotiable as commercial paper is so advantageous to the shipper that such bills of lading have come to be regarded as almost a necessity in any transportation system.

The fourth particular advantage characteristic of the railways is the energetic improvement of their track and rolling-stock as experience and inventive ability, always at the command of large interests, have indicated the successive opportunities for betterment in the interest of economy and improved service. Forty years ago, 56-lb. rails were the heaviest in ordinary use, while now 90- and 100-lb. rails are common, and other details of track and roadbed have been correspondingly improved; then, freight-cars

[1] *Engineering News*, Vol. 63, p. 253.

having a capacity of 10 tons was the rule and the weight of the car about equaled its load, while now 30- and 40-ton cars form a large proportion of the rolling stock, and the weight of the car itself has been reduced to half its load-capacity; a notable locomotive of 1876 weighed less than 100,000 lb., while now one approaching 400,000 lb. is less noteworthy, and their tractive power has been correspondingly increased. Extensive and expensive improvements to secure greater safety have been progressively introduced, such as air-brakes, various effective signal systems, automatic couplers, etc.; and such unifying developments as the changing of the gauge of thousands of miles of southern railways, from one of 5 ft. to standard gauge in 1886, thus permitting cars to reach any part of the country and so avoiding the delay and expense of transferring their freight, have further added to the efficiency of the railways. Such effective progressiveness has given to the railway service a deserved prominence.

Transportation on inland waterways, on the contrary, handicapped as it often is by such natural impediments as low water, ice, vulnerability to storms and uncertainty of the navigable channel depths, and by such financial restrictions as result from the comparatively meager capital and disorganized condition of the numerous small companies engaged in this business, has failed to develop many details of service that are essential to a successful traffic. Terminals and their economic freight-handling equipment are almost unknown, except in a few cases; through bills of lading are rare, except where the railways also control the steamboat companies; the shipper must insure his consignment to secure himself against possible loss, and agencies for the advertising, soliciting and booking of business exist at only few places. Although there is a slight indication of improvement, for the most part the river boat is typically the same as it was more than a half century ago. To increase the economy of transportation by enlarging the capacity of the boat is dependent upon expenditure to deepen the channel, which is a vastly greater expense than to improve the track to accommodate heavier rolling stock. However, the tonnage transported by a single steamboat in channels of restricted depth is sometimes greatly increased by the only notable development in river transportation that can be mentioned, the lashing to it of loaded barges to tow them to their destination.

8. The Present Situation in this Country is not Normal.—This being the actual situation in this country at the present time, many believe

that the improvement of the waterways would be a mistaken policy; yet this conclusion would be intrinsically an error, for three fundamental reasons: 1, present conditions are not, in general, a result of free relative development of railways and waterways; 2, where density of traffic and waterway conditions permit, navigation flourishes, as in the Great Lakes, the Erie Canal (now being reconstructed to accommodate boats of greater tonnage in order to reduce cost of transportation), the Hudson and Ohio Rivers, and in several countries of Europe; 3, transportation by water is essentially cheaper than by rail.

To revert to one of the instances of lack of free development of both agencies of inland transportation, the adjustable tariff rates of railways which often ignored the cost basis and were frequently placed at "what the traffic would bear" gave free opportunity to destroy water competition by establishing the competitive rate low enough to accomplish this desired result even if it were below cost, making up the deficit by unfairly high rates either on high-class freight that must go by rail, or on all freight on the non-competitive parts of the system. The persistence of this influence is apparent from the following quotations from the Preliminary Report of the Inland Waterways Commission, 1908:

"The accompanying tables of river rates and rail rates to points affected by river competition show that the water traffic materially affects the rail rates." "An interesting comparison of all-rail rates applying to points on the rivers and those of inland points . . . (shows that) the inland cities of the same distance as those located on the river are given higher rates in all cases." "Upon the rail lines operating from mines in West Virginia to points along the Ohio River the rates to inland points of equal and even less distance are higher in proportion by from 30 to 40 cents per ton" (33 to 50 percent). "On the Columbia . . . the rates applying from the Dalles . . . to Portland, a distance of 87 miles, is 25 cents; to Castle Rock, a distance of 74 miles, is 50 cents. The rate to Astoria, a distance of 187 miles, is 50 cents, while to Ainsworth and Pasco the rate is 90 cents on first-class freight."

Portland and Astoria offer competition by river, while the contrasted cities do not. When the rate on coal by rail from Pittsburgh to Worcester was practically a half cent per ton-mile there was an alternative of shipping by rail to Philadelphia at about the same rate; thence by water a greater distance to Providence at about ⅛ cent per ton-mile; and thence to Worcester by rail again, for which distance the existing conditions permitted the tariff to be fixed at slightly less than 2 cents per ton-mile. That this economic warfare

has been largely responsible for the progressive decline of much of our waterway traffic cannot be doubted when we note that, where the development of inland waterways is not throttled, their service has increased greatly; as in France where the total ton-mileage in 1905 was two and one-half times as much as in 1880, and in Germany where it was more than five times as great as in 1875. Fortunately the provisions of the Mann-Elkins act of 1910 have probably removed from the future the most serious dangers of destructive competition; and, freed from such commercial warfare and adapting to its needs those many improvements of construction, operation and business acumen which conditions require, the deserving arteries of inland water transportation will gradually attain their just place as component parts of the vast transportation system of the Republic.

Some countries of continental Europe have adopted the method of preventing destructive competition between railways and waterways by requiring that the charge by rail shall not be reduced below a rate about one-fifth greater than the water rate; have built up great harbor and terminal facilities, securing advantageous equipment for transfer of freight between train and boat, and thus have substituted for commercial warfare the sound economic principles of coördination and coöperation. Great industrial and commercial centers have developed marvelously in districts so encouraged, particularly through such valleys as those of the Rhine (10 ft. deep) and the Seine (10 ft. deep) rivers, steamships from the coast cities and from England ascending them as far as Cologne (211 miles) and Paris (230 miles). Surely Germany, France and other countries would not continue this definite policy of protection to the inland waterways if it were an artificial stimulus to an intrinsically inferior method of transportation, particularly in Germany where it is to the disadvantage of the state-owned railways. On the contrary, this approximate "20 percent excess" minimum railway rate mentioned will not average to equalize total cost of shipment between competitive terminals, because of the greater distance by water; and it has, furthermore, a fundamentally sound basis of economic justification in the consideration of actual relative cost of transportation by water and by rail. In densely populated and industrially busy Belgium, which does "not attempt by special tariffs or exceptional treatment to attract the traffic from the state-owned railways," this justification appears in the fact that the actual waterway tonnage of that nation is more than four-fifths the total railway tonnage.

9. Transportation by Water is Essentially Cheaper.—In the complex consideration of the advantageous development of inland transportation facilities in a large way, the fundamental proposition is the economic fact that water carriage under favorable conditions costs less than transportation by rail under favorable conditions. The basic principle is expressed in these terms because it is believed to thus convey a clearer conception of the real situation than does the more usual statement that "water transportation is cheaper than rail," a phrasing which would logically lead to the construction of waterways wherever possible instead of only where a balance of benefits will result, and which may be responsible for many such improvements that are economic failures.

To state that the average rate per ton-mile on the railways of France is more than 14 mills, while the sum of the average water rate (6 mills) and maintenance and capital expenses of the government prorated (about 4 mills) amounts to approximately 10 mills; or in Germany amounts for the railways to 13½ mills, while for the water routes it is approximately 5 mills (rate) plus nearly 2 mills (capital expenditures, maintenance, etc.), or about 7 mills altogether; or in the United States averages (1909) 7.63 mills for the railways (which low figure is mainly due to the average much longer haul and to the relatively much smaller proportion of light, high-class freight caused by our large express business, the extensive railway carriage of bulky freight in this country which largely goes to the waterways in Europe, etc.), is certainly suggestive, but is likely to obscure the essential truth in generalities that may be misleading. For example, noting that the average unit cost of freight traffic in France and Germany is nearly double that in this country, it has been argued that those countries should rather set themselves to improving their methods of railway construction and operation so as to reduce the cost of traffic to that existing in America; forgetting that their capitalization per mile is also about double that in this country ($139,290 in France, $109,788 in Germany and $59,259 in the U. S.), as well as ignoring many other differences of condition, such as those just mentioned, that make an equality of rates chimerical.

It would seem that comparative illustrations, chosen from particular instances where both methods of transportation exist under favorable circumstances, will aid toward a more definite apprehension of the real situation. Thus under advantageous conditions of grade and density of traffic, the Chesapeake and Ohio Railway makes

a rate on coal, from the Kanawha fields to Cleveland, of 1.85 mills per ton-mile; the Illinois Central, from western Kentucky to New Orleans, of 2.1 mills; the Louisville and Nashville, from the Kentucky fields to Atlanta, over heavier grades, a rate of 2.65 mills; and the Norfolk and Western, from the Pittsburgh district to Philadelphia over greater grades, a rate of 3.31 mills per ton-mile.

Similarly, the rates on the Great Lakes, accommodating vessels of 19 ft. draught and 10,000 tons capacity, averaged 0.8 mill per ton-mile in 1907 (it was 16 percent less in 1912); to this should be added, in order to fairly compare the real cost of water transportation with that by rail which necessarily includes all capital charges, the corresponding economic burden for improvement and maintenance of harbors and waterways on the Great Lakes. This will err in a way unfavorable to the water traffic, if at all, because the improvement of harbors also benefits the railways reaching them. Disregarding this, the computation involves the total expenditures on the Great Lakes from the beginning of appropriations, $97,791,108, which, at 3 percent, amounts to an annual charge of $2,933,733; the ton-mileage for 1907 exceeded 69,000,000,000, and the corresponding ton-mileage charge would therefore be about 0.04 mill. Adding to this the average rate of 0.8 mill would give a total comparative cost of about 0.84 mill per ton-mile. An estimated expense to transportation companies of water carriage on the Monongahela River, made in 1904, was 1.8 mills per ton-mile; on the Ohio River, in its condition under the original project, not involving locks and dams, with the barges to return empty from Louisville to Pittsburgh, 0.76 mill per ton-mile; for the Ohio River, when improved by locks and dams to a depth of 6 ft., 0.63 mill per-ton mile; in case the Ohio River were improved to a depth of 9 ft. the similar estimate on the 600 miles from Pittsburgh to Louisville was about 0.4 mill per ton-mile, and for the 1360 miles from Pittsburgh to New Orleans, 0.39 mill per ton-mile.[1]

To make fair comparison of these figures just given with railway rates, there should be added to them a percentage for profit (say 20 percent) and a prorated charge for cost and maintenance of improvement. In the case of the Monongahela River, the Report of the Chief of Engineers for 1910 gives the various expenditures as $7,660,374, which, at 3 percent, would amount to an annual capital charge of $229,811. The same report states the tonnage of the

[1] Lieut.-Col. Sibert's "Memoranda Relating to the Cost of Transportation on the Monongahela, Ohio and Mississippi Rivers."

river for that year as 11,486,278. The average mileage of this traffic is not stated, but assuming this at four-tenths of the navigable length of the river, the prorated capital charge is 0.38 mill per ton-mile. Twenty percent of 1.8 mills is 0.36 mill, and the sum of the three gives about 2.5 mills per ton-mile as the estimated comparative water rate on the Monongahela.

Similarly, the same report gives the total expenditure under the original project for the improvement of the Ohio River for almost 1000 miles (instead of the 600 miles of the above estimate) as $7,314,-153, which, at 3 percent, amounts to an annual charge of $219,425. The commerce of 1910 amounted to 8,676,701 tons. Making the same assumption as to the average length of haul, the prorated charge would be about .06 mill, the 20 percent profit would amount to about 0.15 mill; and these amounts added to the carrier's estimated expense of 0.76 mill aggregate a trifle less than 1.0 mill per ton-mile as the comparative estimated water rate on the Ohio, unimproved by locks and dams. As the estimated amount for the completion of the improvement of the Ohio River to a navigable depth of 9 ft. by the construction of locks and dams, etc., is given as "indefinite," similar estimates of total comparative rates by water transportation cannot be made.

The actual cost of transportation on the old Erie Canal in boats of 240 tons and 6 ft. draft is stated to be 1.75 mills per ton-mile; and the expected cost when the enlargement shall be completed to accommodate boats of 1000 tons is given as 0.52 mill per ton-mile.[1] Fifteen years ago the average river rate for a series of years upon wheat from St. Louis to New Orleans was 1.33 mills per ton-mile, including transfer to ocean steamships at the latter city. Coal has been shipped from Pittsburgh to New Orleans at a rate of less than 0.4 mill per ton-mile.[2]

The reason for this very low river rate (which equals that offered for westward freight on the Great Lakes by vessels returning light for their eastward cargo, and which seems to be exceeded only on the ocean) is the great advantage of loading the freight on barges of moderate draft and combining them into a fleet lashed to the single towboat which controls the movements of the combination. This system is now characteristic of the Ohio River below Louisville and of the Mississippi from the mouth of the Ohio to New Orleans in handling large shipments economically. The number of barges

[1] Report of the Committee on Canals of New York State, 1900.

[2] House Doc. No. 492, 60th Congress, 1st Session, p. 112.

in the fleet of course depends upon the amount of freight to be transported. Often there are thirty barges of 1000 tons capacity, and sometimes as many as sixty, the latter number covering a water area of more than 1000 ft. in length and 300 ft. in width. Naturally these fleets require a very ample width of channel for maneuvering through a winding river and in passing others as they are met, a liberality of space which must increase where the current velocities are great or irregular. Such a shipment as 60,000 tons under the power of one towboat equals that of five or six of the steamboats of the Great Lakes, and exceeds that of the largest ocean steamship afloat. The horse power required for such towboats is only about 2000, while that of the steamship "Imperator" is 62,000; and the corresponding cost of the boats per ton of freight carried is given as about $12 and $70, respectively. Such contrasts in favor of the river boats go far to offset many disadvantages; and these, together with the evolution of the barge-fleet formation, which allows the controlling power boat to transport a great tonnage by spreading the load over a wide area where the navigable depth is comparatively small, signifies much for the economics of river traffic.

In comparing these instances illustrating particularly favorable cases of transportation costs by rail and water in this country, chosen as far as possible to represent such typical conditions as would aid in forming a just comparative impression of relative economic expense under favorable conditions for both, the persistently lower cost of inland water transportation is apparent. This fundamental proposition is not only corroborated by the experience of several European countries, as cited, but by such incidental experiences as that of the Chesapeake and Ohio Railway and the Ohio River steamboats agreeing to prorate on the basis of 2 miles of waterway transportation balancing 1 mile by rail. On the Rhine, transportation by river is stated to be from two to three times as cheap as by rail, for distances exceeding 300 miles.

10. The Significance of Density of Traffic, Terminal Facilities, Etc.—It is regretted that the illustrative examples are not more numerous, so as to allow an approximate average estimate of relative costs, but were this possible it would be of very doubtful advantage because conditions of each case vary so greatly. This is particularly true if the traffic be small, as is the case on most of the American waterways at present, the resulting effect on cost estimates being a relatively very great unit debit for construction and main-

tenance upon the existing traffic, which makes the improvement economically unwarranted until the tonnage becomes sufficiently large; a principle that applies with equal force to railways, or other similar enterprises.

When it is remembered that France had expended (1821–1900), on the construction and maintenance of about 10,000 miles of inland waterways, more than $449,000,000; Prussia (to 1906), on 8750 miles, about $133,000,000; Russia (in the last hundred years), on 22,000 miles, more than $500,000,000; Belgium (to 1905), on 1300 miles, about $101,000,000; and the United States government (to 1911), on 28,600 miles, $383,958,205 by the federal government, and about $214,000,000 by states and corporations,[1] it is very evident that a heavy traffic is necessary in order that the fixed charges as theoretically prorated upon tonnage carried shall not be excessive. In fact, so serious has been the discrepancy between expectation and fulfillment that, of the mileage statistics given above for Germany and France, nearly 30 percent is "not available for navigation"; and this country has, in the last three decades, abandoned nearly 3,000 miles of canals and canalized rivers on which there had been spent more than $70,000,000. The average density of railroad traffic in the United States exceeds 1,000,000 ton-miles per mile per year, while in the case of some lines it reaches four and five times that amount. The average density of commerce over our improved waterways is not known, but it is probably less than 100,000 ton-miles per mile per annum. In France the corresponding figures are about 400,000 for the railways and 400,000 for the waterways; in Germany, 800,000 and 1,500,000 respectively; and in Belgium, 550,000 for the waterways. In view of these general statistics, the relatively flourishing condition of transportation on European waterways, and its general unsatisfactory condition on our own, is not remarkable. Still more evident is this influence when we note the high relative density on those waterways which are notably serviceable in commercial activities; the figure for the German Rhine on the 265 miles between Mannheim and Emmerich exceeded 20,000,000 tons in 1910; for the Seine between Paris and Rouen, 3,300,000 tons; the Hudson, about 18,000,000;[2] the Great Lakes, more than 50,000,000; the Ohio, 2,800,000 and the Monongahela, 3,600,000 ton-miles per mile per annum.

[1] From "Preliminary Report, U. S. National Waterways Commission," p. 30, McPherson's "Transportation in Europe" (1910), and private information.

[2] S. Doc. 301, 61 Cong., 2d Sess., p. 48.

Perhaps an equally important administrative consideration is that of facilities for the economical handling of the freight. So serious is this burden upon our railways that, even with the study and attention which they have devoted to this question, the terminal costs in the United States have been said to approximately average an amount equivalent to the cost of 250 miles of actual haul over the tracks. With regard to our waterways the situation is far more serious, because such facilities are generally most crude and uneconomical except in the case of transportation companies owning large fleets and commanding a large business; like those owning the iron-ore docks on the Great Lakes or the coal docks of the Ohio and Monongahela Rivers. In order that the freight terminal costs for loading and unloading the boats shall not be an extravagant amount, so making the total expense of water carriage excessive, it will be generally necessary for municipalities in their corporate capacity (or else private corporations formed for this purpose) to establish adequate facilities for the effective handling of general freight wherever commercial conditions warrant this procedure, which shall be available to any carrier meeting the necessary charges. Such terminals must offer convenient and adequate railway connection and transfer facilities, as well as the simpler accommodations for wagon transfer; and they must be very definitely so situated, controlled and administered that the greatest freedom of commercial movement shall result, allowing traffic to follow whatever route or line is economically advantageous for each commodity, unhampered by restrictive or coercive influences of rival interests. The seriousness of the situation may be judged from the statement that, even in the case of the comparatively efficient port facilities of Chicago and Buffalo, approximately one-third of the total cost of shipment is absorbed at each of these transfer harbors, while the actual movement through the lakes between those cities (885 miles) costs the steamboat company only about the remaining third of the total freight charge.

European experience in waterway transportation has been particularly productive of extensive and splendidly equipped terminals on those channels whose commerce has grown the most rapidly. The Rhine has more than sixty such interior harbors for the loading, unloading and storage of freight, at about two-thirds of which railway connection and transfer facilities exist. The general plan of that at Neuss is shown in Fig. 1 (p. 20), and a view of one of its basins is given in Fig. 2 (p. 21). The arrangement of the still more

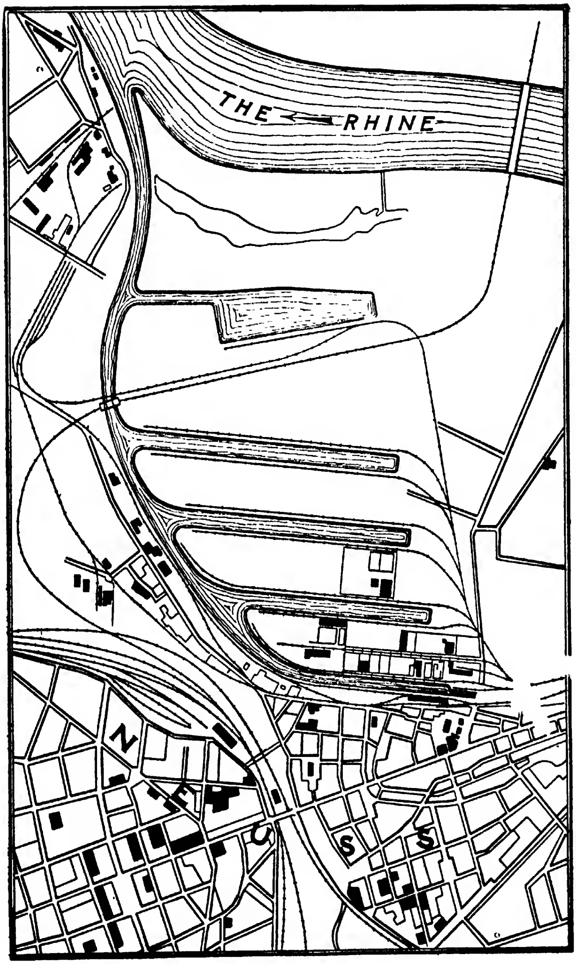

Fig. 1.—A river harbor.

extensive harbor of Düsseldorf is indicated in Fig. 3 (p. 22), and the transfer facilities existing at the Berger basin are suggested by Fig. 4 (p. 23). A general description of the important features of a transfer harbor, so essential to permit the railway collection and distribution of river freight to the extensive territory back from the river, is quoted from "The Navigable Rhine" by Edwin J. Clapp:

"A first-class river habor must have first of all a water area so great that many boats can come in and out, can be loaded and unloaded at the same time without disturbing each other. In winter the dues of the swarm of boats hibernating in a large harbor are a good source of revenue. This

FIG. 2.—View of a harbor basin.

water area is usually provided in the form of basins; the tongues of land that divide them carry railroad tracks. Perpendicular quay walls allow the boats by low water as well as by high to lie close in, within reach of the cranes. For discharging package freight cranes are needed; elevators discharge grain. Coal is unloaded into the freight car by a crane or distributed by the self-loading bucket of the traveling loading bridge over the coal magazines that line the harbors on the upper Rhine. Floating cranes transfer goods from one barge to another. On the quays run double railroad tracks so that cars can be loaded and switched out at the

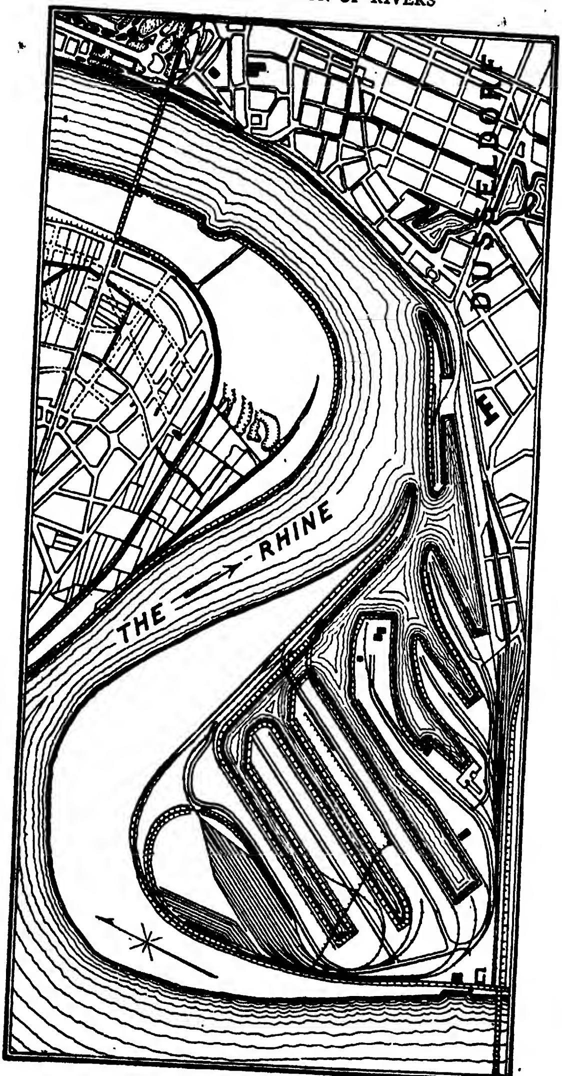

Fig. 3.—An extensive river terminal.

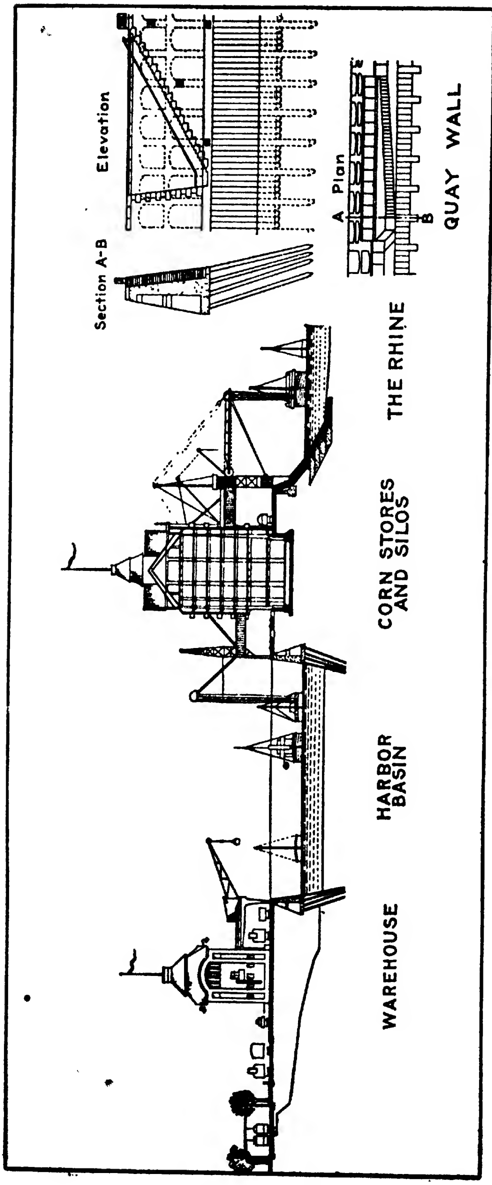

FIG. 4.—Typical transfer facilities.

same time. The portal cranes that span the tracks load into cars or swing the goods across to the first or second floor platform of the sheds beyond. Customs officials must be on hand to expedite dutiable foreign wares. The local merchants demand warehouses at the waterside with a department for bonded goods for which the duty need only be paid when they are taken out to be sold. Dealers in grain erect silos on the water's edge, which by means of grain elevators fill themselves direct from the barges. To shelter the shipments of the Standard Oil Company, the harbor has a tank plant, isolated from the rest of the harbor, with a capacity of thousands of tons of petroleum. Oil is pumped from the barges into the tanks, where it awaits distribution through the city in an oil wagon or re-shipment inland in a tank car. Lastly, no ambitious Rhine city can be without an industrial harbor—a block of land attached to the commercial harbor which is offered for sale to industrial concerns. At first it was the iron industry, then the chemical, that settled in these harbors, as they received barge-load consignments of raw materials up the Rhine. But such is the attractive force of cheap water rates that to-day in the Mannheim industrial harbor, for instance, all sorts of industries are represented, from steam flour mills to a mirror factory."

So essential are such adequate terminals that several German cities have spent many millions of dollars each on extensive installations of this kind; one of these transfer harbors, that of Duisburg-Ruhrort, had in 1907 a terminal tonnage almost as great as the sea traffic of Hamburg.

The general situation existing in this country is well summarized in House Document No. 50, 61st Congress, 1st Session., in stating:

"The only people in the Mississippi Valley who have paid much attention to terminal facilities for freights are the railroads; and consequently they command the transportation business. Individual boats and small corporations cannot afford to pay for such terminal facilities and the cities and towns have either not yet seriously considered the question of establishment of such facilities or are not prepared to pay for them. Even on the Great Lakes, the railroads so dominate this situation that for the past many years only one out of many steamboat lines has been able to stand alone without being under the control of railroad terminals, and it is the water rates which are now being gradually raised to equal the rail rates, instead of the rail rates being lowered to or controlled by the water rates. The supremacy of a port depends mainly upon its ability to exchange land and water commerce easily, readily and economically."

Density of traffic is, again, a great factor in reducing the relative burden of terminal expenses.

It therefore appears that water transportation is economically advantageous where there exist certain favorable conditions, such as a route capable of adaptation to this use at reasonable expense, freedom of commerce to seek the intrinsically advantageous route and facilities to permit this, and a density of traffic sufficient to reduce the fixed charges to a small unit rate. There is always necessary a thorough and expert analysis of the conditions affecting each possible route which becomes economically available as growth of population and increase in commercial activities bring each project into consideration. The especial service of inland waterways is the transportation of heavy, bulky, imperishable freight, such as grain, coal, ore, building materials and other raw materials, etc., over these natural arteries of commerce; leaving to the railways the transportation of the higher priced freight in the form of perishable goods, manufactured products and all those commodities for whose delivery a greater speed or directness is desired. Yet there is good reason to doubt the conspicuous superiority of the railway in general capacity to quickly deliver freight when it is noted that the average daily freight car mileage of this country for the years 1907–13 was only a fraction over twenty-three. In a region of notable commercial activity where the conveniences and cost of transportation are a vital factor in the development of its prosperity, the railway and the waterway should be in no sense antagonistic competitors for exclusive consideration, but rather should be constituted complementary agencies of effective intercourse; each one performing that particular service for which its special characteristics best qualify it, and so contributing most efficiently to the industrial prosperity of the community. As a result of careful study and experiment, electric hauling is beginning to supplement the railway service as offering particular economies under special conditions. Everything considered, we have not yet learned to differentiate enough in our transportation problems. That this coördinated relation between waterways and railways is the fundamentally true one is indicated by experiences such as the prosperity of both rail and water lines in regions of present commercial activity, as from Duluth and Chicago to Cleveland, Buffalo and New York; from the Pittsurgh region to Cincinnati, Louisville and the southwest; from New York City to Fall River, Boston and beyond; and on the Great Kanawha, while the river tonnage increased from 9,500,000 before the improvement of the river to 26,500,000 tons after that accomplishment, the railway tonnage in the same time grew from 6,500,000 to 31,000,000 tons per

annum. As the need of the most efficient development of transportation facilities shall become more fully recognized as an essential part of the great industrial growth of the coming years, undoubtedly the balance of advantage will, in many cases, be found to favor the waterway as a vital factor in the commercial achievements of the unfolding century.

In regard to the general question of coördinating rail and water transportation facilities of this country in order to secure to the public a maximum of economic advantage, the Final Report of the National Waterways Commission recommends that water lines engaged in interstate commerce should be made common carriers in the legal sense so that the Interstate Commerce Commission would have the power to establish through routes so as to include water transportation for such part of the route as might be advantageous, which would not only render waterways available for commerce originating near their course but would also vastly extend the territory subject to their influence; to require joint rates; to secure through bills of lading to include the water routes; and to regulate the charges by water as well as by rail. The requirements of efficient regulation of port and terminal facilities are thus essentially summarized in Paper 21 of the International Congress of Navigation (1912).

"Whether terminal and intermediate ports are developed by private interests or by the municipalities, it is essential that each port should be systematically organized for the accommodation of the traffic and the industries to be served. In some instances, this has been brought about by public regulation of ports owned and developed solely by private capital; but experience conclusively shows the need of supplementing public regulation of privately developed terminals with the municipal ownership and operation of wharves, docks, warehouses, and other harbor facilities for the general use of the public. The number and variety of wharves and other facilities that should be maintained by the state or municipality at any particular port will depend upon the local requirements of the port. Exclusive private ownership of water terminals is indefensible.

"The actual legislative and administrative measures to be taken to coördinate railroads and waterways, to unify and systematize port facilities and to provide an efficient harbor administration must vary with different countries. In the United States and countries having similar political organization it is necessary that the Federal Government, which has authority over interstate commerce and carriers, should require railroad companies engaged in interstate commerce

1. To make physical connections with waterways.
2. To exchange traffic with the waterways.
3. To issue through bills of lading and quote through rates over combined rail and water routes, and
4. To secure to shippers the option of dispatching freight by an all-rail or by a rail-and-water line, when a choice of routes is possible.

"The physical layout of intermediate and terminal ports and the mechanical appliances best adapted to the handling of traffic must be determined for each port separately and in accordance with its special requirements. Local city and state engineers must apply to the solution of local problems, and adapt to local conditions, the principles of port organization and operation that have been found effective at other ports and in other countries."

11. The Improvement of a Waterway is an Economic Question.— To predetermine whether the improvement of a waterway for navigation will be economically justified by results calls for the same insight and acumen which is needed in estimating the outcome of any business or commercial venture, though the benefit or loss is national rather than personal; and it also requires that broad judgment and thorough knowledge of waterway improvements which will enable the civil engineer, not only to estimate closely the cost and service which the inauguration of a particular project will involve, but to determine which of several possible projects for an improvement is best fitted for the particular conditions concerned in the proposition. He will determine the present commerce, and he will estimate (from present conditions and tendencies) what its character and tonnage will be at a future date that is found to be advantageous; because reconstruction and enlargement of works of improvement are so troublesome, expensive, and sometimes destructive of previous work as to make it economical to definitely anticipate future needs. From detailed surveys he will estimate the costs of alternative projects, differing both as to method of improvement and as to depth of waterway to be provided. This consideration of the different methods of improvement and their particular applicability to the varying conditions of the waterways are the especial purpose of this volume, and will be discussed in detail in succeeding chapters; but the question of what constitutes the most advantageous depth is a preliminary proposition, and is therefore outlined here.

The cost of the improvement increases rapidly with the depth to be secured. Any river has some navigable depth which, if sufficient for commercial needs, requires no expenditure. A somewhat greater

depth involves a deepening and perhaps a widening at certain places, which become rapidly increased in number as well as in dimensions for each additional foot of depth desired, if the improvement is made by the process of excavating shoal places; and, if effected in other ways, the proposition is similar in principle; that the cost increases very rapidly with the depth obtained. This ratio of variation is necessarily different in each case considered; but it averages, perhaps, more or less as the third power of the ratio of the proposed depths to the naturally navigable depth.

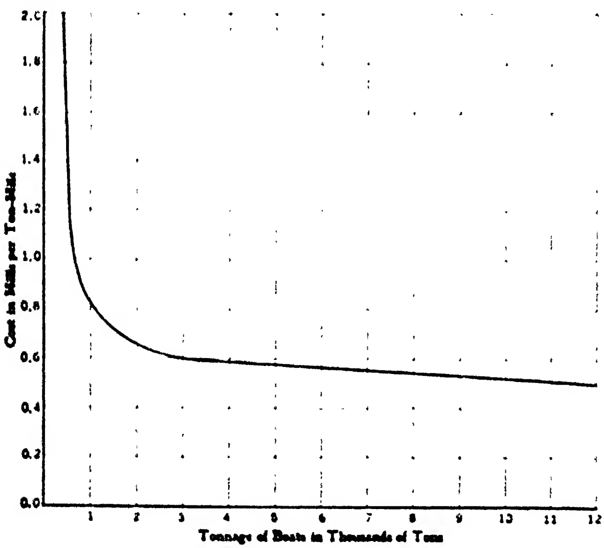

FIG. 5.—A cost-tonnage curve.

On the contrary, the economy of transportation increases notably as the tonnage of boats increases; and further the ratio of tonnage to draft, or to depth of waterway, is also a rapidly increasing one. Again, both these economies follow no definite mathematical law; but a typical diagram (Fig. 5) is given illustrating suggestive average values of the ratio between cost of transportation and the tonnage of the boats used; while the second diagram (Fig. 6) similarly shows roughly the relation between draft and tonnage of boats, as averaged from a large number of transportation routes.

Of course, these two diagrams can be united into a single one (Fig. 7, p. 30) showing directly a crude, but typical, relation between draft and cost. For example, if it is desired to make a preliminary estimate as to whether it will pay to deepen a natural channel of 6 ft. to one of 9, when the estimated tonnage will be 2,000,000 per mile, and assuming that Fig. 7 represents the relative cost values at the locality in question, we find that for a draft of 5 ft. the cost per ton-mile is 1.70 mills, and for 8 ft. it is 0.82 mill. The

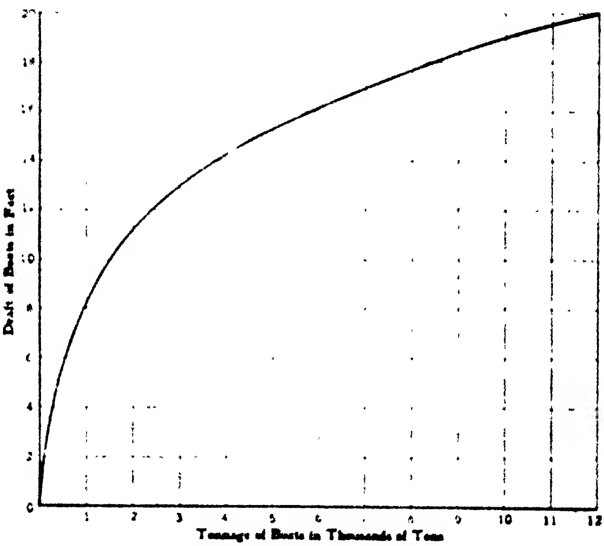

FIG. 6.—A draft-tonnage curve.

difference multiplied by the estimated annual density of traffic gives $1760, which is directly justified as an annual expense per mile for this improvement; while if permanent works of regulation are planned, the interest charge being estimated at 3 percent and maintenance and depreciation at 2 percent, an average initial expenditure not exceeding $35,200 per mile is allowable. The final figure of this supposed example would be increased by the estimated value of the collateral benefits mentioned in the latter part of this chapter.

This example and the diagrams merely suggest the basic principles;

they cannot be considered mathematically accurate, nor even necessarily approximate, as they may be in error 50 percent or more when applied to a particular case because of the variations in kind, dimensions, motive power and speed of boat, facilities for freight-handling, and other factors whose influences are averaged. For example, the custom of largely overcoming the disadvantage of deficient channel depth by uniting a considerable number of barges into a single fleet towed by one power-boat, would greatly change the curves of the diagrams. However, such special diagrams, al-

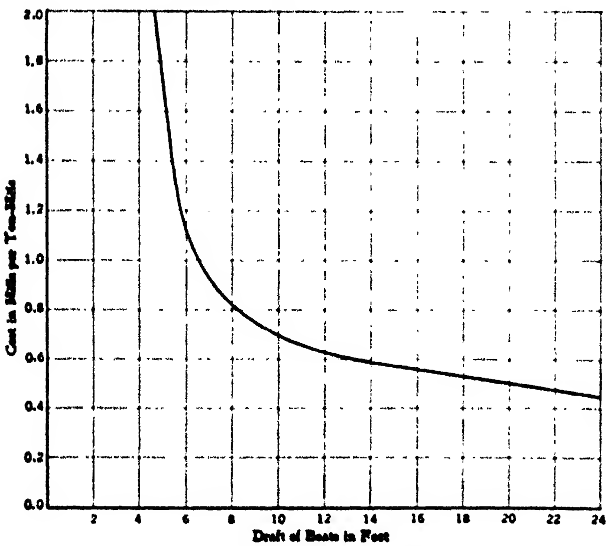

FIG. 7.—A cost-draft diagram.

though differing in position and curvature, would undoubtedly be similar in general form to those here given. This discussion, then, simply illustrates the economic law which is to be applied in such an investigation; the relative values which shall be used in each case are to be derived in conformity to the particular circumstances and conditions. In any case there is a certain navigable depth to which it is economical to improve the waterway; for less depth than this the cost of the works is justified by the resulting economies of transportation, but for greater depths the costs increase faster than those

economies. It is the duty of the engineer to determine this economic depth for each project in relation to the cost, both of construction and of operation, under the conditions of transportation which will there obtain.

A correlated consideration, which may modify the conclusion just discussed, is that of the general economy which follows the standardization of connecting waterways to a maximum extent so that boats may pass from one to another in response to the requirements of commerce, instead of making necessary the transfer of goods to boats of a less draft. The adoption of this policy largely accounts for the prosperity of European waterways. France and Belgium have adopted the 300-ton barge, drawing 5½ ft., as the general standard, and the Prussian waterways are, in most instances, being adapted to barges of 400 to 600 tons, drawing 7 to 8 ft., in the territory east of Berlin, while the tonnage of the barges in the provinces to the west of that city is from 600 to 800, having 8 to 9 ft. draft. It is apparently in view of the frequent inadequate consideration of the inherent economic advantages of a studied standardization of depths of inland waterways of this country which led the U. S. National Waterways Commission (1910) to state:

"There are manifest benefits in securing standardization or equal depths in all channels, so that boats of the same size may be utilized everywhere. This would prevent the very considerable expense of loading and unloading whenever there is a transfer between channels of different draft or capacity. If a uniform depth can be secured, especially in locks and improvements of artificial construction, it should be accomplished. The fact, however, must not be overlooked that the very great difference in the size of the streams which make up any river system renders complete standardization difficult and exceedingly expensive, if not impossible."

12. Collateral Benefits are also Important.—Such are the principal direct economic considerations involved in the improvement of interior waterways. There are frequently, however, collateral advantages whose real benefits redeem an otherwise unjustifiable expenditure, and occasionally even outweigh in importance the direct interests. That these advantages are more vague and their definite estimate is more difficult than are the direct ones, constitutes no valid reason for ignoring or slighting them. On the contrary, the greatest care and foresight should be brought to bear upon the determination of these abstruse but actual advantages in order that

their benefit may be measured and given its proper weight in the consideration.

The most usual and obvious of these collateral benefits is the reduction in the cost of transportation which generally accompanies waterway development. The marked difference in rates so often existing between railways competing with waterways and those which do not, examples of which have been cited, is evidence of this regulative effect. Further, water rates have very generally been a material factor in the determination of railway rates when waterway traffic preceded the railway, as already noted; and when river navigation has followed the rail it is no less usual to find that the railway rates have been notably lowered. For example, the report of the Chief of Engineers, U. S. A., for 1910 mentions many different cases such as the following:

"While it is not claimed that river improvements are solely responsible for the great difference in freight rates as shown, it is well known that wherever water competition exists, whether by river, canal, or lake, its effect on freight rates is always beneficial to the public, and that so long as the navigation of the Mississippi River is feasible, and largely in proportion to its feasibility, such benefits will accrue even if but little river commerce is actually carried on. . . . . It is well known that the railroads strive keenly for the north and south freight business, making much lower rates where there is water competition, cut their rates on certain commodities, and, it is reported, make sometimes especially low rates for the season of navigation, to be increased during the winter months. The latter practice was common several years ago." "The closing of the (Illinois) river to navigation would unquestionably lead to a rise in freight rates on the adjacent railways, and as the volume of freight affected would be large, a good navigable condition should be maintained." "The improvement (of the Mokelumne River, California) has resulted in a reduction of freight rates." "Freight rates are said to be already considerably lowered by this (Tennessee) project." "As nearly as can be determined, the effect of the (Ocmulgee) improvement has been to cause a reduction of from 25 to 40 percent in freight rates." "It is reported that the Tennessee Central Railroad, which parallels the Cumberland River from Nashville to Clarksville, had rates in force before the completion of Lock and Dam A of from 18 to 26 cents per 100 lb., and since this lock was put in operation these rates have been reduced above the lock to from 6 cents to 12 cents per 100 lb." "It is considered that water transportation has aided largely in the development and prosperity of the contiguous country, and has materially affected freight rates." "The effect on freight rates of the slack-water system of the Monongahela is very great. This is particularly true for coal, which is the principal article of com-

merce, and is well shown by a comparison of the railroad rates for carload lots along this river and those along the unimproved Allegheny. There are many mines along the Monongahela River that can ship either by rail or water and within a distance of 45 miles from Pittsburgh, which practically covers the industrial district on that river, there is a rate of 10 cents a ton on hauls not exceeding 7 miles. For corresponding distances on the Allegheny the rates average about 35 cents a ton. One large consumer transports its coal by river, a distance of about 50 miles at a total cost of less than 10 cents a ton, including all charges, while the corresponding railroad freight rate is 45 cents a ton." "The improvement (of the Coosa River) has resulted in a reduction in railroad rates between points on the river and either Rome or Gadsden of not less than 50 percent, water rates controlling all shipments to and from the country contiguous to the river." "The effect of the (Missouri River) improvement has been to equalize and keep down freight rates, the actual river rates being about 60 percent of the railroad rates."

On the Columbia River, the Cascades improvements seem to have been particularly advantageous indirectly; for the freight rates between Dalles and Portland, Oregon (over the railway following the south bank of the Columbia River) which had been held at $6.20 per ton previous to the opening of navigation, were suddenly reduced to $2.00 per ton when the completion of the canal made river transportation possible. This reduction in freight rates is stated to have saved $7,000,000 to the people of Dalles in the first ten years of navigation through the Cascades. It is very true that railways competing with waterways are generally less expensive to construct and to operate, because of more advantageous topography and lower grades, than are lines away from navigable river valleys; but this accounts for only the smaller part of the difference in rates. Railway officials have repeatedly acknowledged this influence and to its legitimate extent this public benefit should be credited to the waterway.

Among the various uses of water for public purposes, it occasionally occurs that the interests of irrigation or of water power development are invoved where propositions for navigation are concerned; more often are questions of the drainage of lands and of protection from floods involved with waterway improvements. Whenever any such public services are aided through the construction of works for improving navigation, this value should also be credited to the improvement of the waterway.

Even more difficult to definitely apprehend, but nevertheless often important and sometimes of great moment, is the particular influence

of waterways upon the growth of population, industries, property values and taxable wealth which results so naturally, in regions of natural resources enjoying a maximum of economic freedom, where commercial facilities are wisely developed in a way to form the most adequate transportation systems obtainable. The wonderfully rapid industrial progress of Germany has occurred simultaneously with her notable development of waterway transportation facilities, enabling inland regions to compete successfully with great industrial centers which were naturally situated more advantageously in the strategy of the world's commercial rivalries. The country tributary to the improved river Rhine has rapidly grown in industrial importance, until it now produces more than 60 percent of the total manufactures of the German Empire. Much the same amazing industrial progress has occurred in the interior of Belgium; and in France, along the Seine and from Paris northeastward to the frontier, the business activities and transportation facilities both by water and rail have kept fairly equal pace in the march of progress. In this country, the marvelous economic growth of the Pittsburgh region has been accompanied by adequate expansion of rail and water facilities for the transportation of ore, coal and other commodities; and other centers of intense activity are experiencing a similar development. It is by no means intended to attribute the material progress of such industrial centers wholly to waterway development; nor is it to be supposed that such transportation facilities are without influence. As well might we expect to prove either that one's physical powers result only from an excellent circulatory system; or, as the contrary extreme, to insist that a good condition of arteries, capillaries and veins contributes nothing to robustness of body. When constructed in response to commercial needs, the arteries of commerce are as truly an item in industrial progress as are the arteries of the human body an essential factor of physical vigor. In estimating the value of a waterway it is, then, only just that due credit should be given for whatever amount of real industrial growth is justly attributable to the influence of its operation.

Undoubtedly the indirectness of benefit and the difficulty of the analysis necessary to correctly estimate the effect of a waterway improvement upon the general prosperity of a tributary territory, leads frequently to absurdly exaggerated claims, and also often results in the opposite error of entirely ignoring such benefits. Either extreme conclusion is to be deprecated as an omission of an essential factor in the case, through a superficial or indolent attitude

toward it. The very indirectness and vagueness of the proposition necessitates the most careful study and profound reasoning power; and whether these estimated advantages be small or great, wholly lacking or even paramount, their determination surely is germane. So pervading is the influence which great public works exert indirectly upon the general welfare, that the discriminating investigator will discern and evaluate these benefits even as the detector of the wireless telegraph system selects and responds to those elusive ether waves which it was so skilfully designed to reveal. A penetrating analysis, a constructive insight and a clear vision must all unite to secure that completeness of estimate of both direct and indirect benefits, which permits a just judgment of the merits of a project, the subsequent realization of which proves the wisdom of the enterprise.

CHAPTER II

GENERAL PHENOMENA

13. The General Nature of Rivers.—A keen apprehension of the natural condition of rivers and the laws of stream flow is an essential preliminary to the consideration of their regimen. The successful improvement of a river is fundamentally dependent upon a thorough understanding of all those phenomena, both general and particular, immediate and remote, which contribute to that resultant condition of the natural stream which is to be artificially modified for the purpose of securing its effective navigation.

The flow of streams is primarily a physiographic function of the drainage of the land areas of the world, carrying the surplus waters which fall as rain or snow ever onward toward their ocean goal; this onward progress, from their sources toward the sea, is typically marked by a general increase in volume and decrease in slope. For example, the Cumberland River, which is about 700 miles long, flows through the upper one-fourth of its course on an average slope of about 7 ft. per mile, though the slope near its headwaters is several times this rate and is correspondingly less in the lower portion of this section; the middle half of the river has an average slope of about two-thirds of a foot per mile but typically varying from this in various sections, and the lower one-fourth of its length is on a varying slope which averages about four-tenths of a foot per mile. Similarly the volume of low water flow is very small at the headwaters, gradually increasing toward the head of navigation, but through the greater part of this upper quarter of its length it "shrinks to a mere brook during extremely dry seasons." The river is navigable for about three-fourths of its length, the volume in this portion being many times that in its non-navigable part, and also increasing toward its mouth. The volume is fundamentally a function of rate of rainfall and of area above the place considered; and the general slope is essentially a question of the topographical conditions of the valley. Both the increase of quantity and reduction of slope have the general effect of enlarging the area of cross-

section of the river. It is only when the volume becomes sufficiently augmented and the slope enough reduced, with the accompanying enlargement of section and with a favorable velocity, that navigation is practicable.

The facts just outlined lead to the theoretical division of a river into its navigable portion and its non-navigable headwaters which, with the tributaries of the navigable part, are technically considered merely as "feeders" to the latter, the detailed study of which is the purpose of this volume.

The ideal for a navigable stream would be a constant velocity. This would not only make more favorable and safe the navigation of it, but would especially secure that stability of channel conditions whose lack constitutes the principal defect of natural waterways. The correctness of this proposition is generally corroborated by observed facts, except where particular influences counteract its effect; for, the less the variation in the velocity of a stream, the less violent are the changes in depth, width and position of channel.

That the mean velocity of a river varies so much at different times, at any place, is due to various natural causes; principal among which is fluctuation in volume of flow. At times when rains or melting snows do not occur on a watershed, the run-off of the stream becomes small; and the longer this condition continues, the more reduced will be the flow until, in times of drought, the volume and velocity, depth and width, reach their minimum. On the contrary, when rains are numerous or heavy the volume of run-off is increased; and it will reach a relatively large amount when these causes are sufficiently intensified. In the cooler regions the melting snows contribute their waters when the spring temperatures thaw the accumulated deposits, the result being particularly evident if the thaw is accompanied by rain. In either case the effect upon the increase of volume and velocity of the river is particularly pronounced if the ground happens to be saturated or frozen; while the opposite effect results if the movement of the surface waters to the stream is delayed by absorption of the soil or by retarding surface conditions such as forests, swamps, marshes and lakes.

Remembering then that the ultimate origin of the river waters is precipitation, which is itself so extremely variable in locality, intensity, duration, and frequency; and noting that the conditions just mentioned, which principally influence the rate at which the surface waters reach the feeders, may at the same time exist in all varying degrees of occurrence, extent and retarding effect; and,

further, considering that the headwaters and branches of the navigable river may (at any one time) be contributing to it very differently in quantity, the flow from each corresponding to the conditions then existing upon its own watershed so that the aggregate volume may sometimes equalize through a balancing of various high and low stages of the streams, while occasionally it may reach a maximum through the cumulative effect of flood waters from each feeder happening to coincide with that of the others, or a minimum stage occur under opposite extreme conditions; it seems in no way strange that rivers are subject to such great fluctuations in depth and width, volume and velocity, the minimizing of which is so desirable to the interests of the river valleys and so important to navigation. For example, the Engineering News of March 24, 1910, thus discusses the causes of the great Paris flood of that year:

"About Jan. 15, we have the following condition: the ground near the Marne and its branches saturated with water so that it could hold no more, the sub-soil at Paris in a saturated state and the plains and mountains of Morvan covered with snow. At this time a thaw set in on the Yonne and rain commenced to fall there, and the run-off rapidly reaching the Yonne, started down the river toward Paris. From then on until March, warm periods, with rain and consequent thaws, alternated with cold spells which stopped the excessive run-off for a time. About this same time, too, the sub-surface in the northeastern branches of the river had about reached a point where it could hold no more water and the run-off into the stream began to approximate the rainfall in amount. Beginning on Jan. 19, the floods from the fast rising Yonne began to arrive and swell the river already rising from the general run-off from the rain-soaked lower river. These floods from the Yonne increased until about Jan. 22, when they were reduced by the freezing up of the mountain district on Jan. 21 and 22. It seemed at this time at Paris that a fall of the flood height could be expected when the water from the slowly moving Marne and Haute-Seine reached Paris some four days after the floods from the Yonne caused by the same conditions over the whole area. Again on Jan. 23–25, another warm spell appeared on the Yonne and its rapid run-off and flow precipitated another flood on Paris, joining therewith the steady flow now coming down from the Marne. These last two combined on Jan. 28 to form the crest of the flood. . . . The cause of the flood was, then, the recurrence of warm weather and rains which allowed the concurrence of floods from two rivers which ordinarily would follow each other. The great height of the water at Paris was due also to other causes, namely, the saturated condition of the sub-soil around about the city due to a rainy and snowy winter, the very gentle slope of the Seine from Paris to the sea, and the excessive number of bends in the river (both of which prevented

the water from flowing away rapidly), and finally by the great number of obstructions to the river's course in the way of bridges in the city proper."

14. Rainfall and Run-off.—The retardation of precipitation, in its movement toward the streams, is the principal protection against the troublesome flood and low water extremes of rivers. Nature has furnished various agencies whose passive constraint is interposed to reduce the severity of such fluctuations; and while, as a rule, only the smaller part of the total rainfall finally flows away in the main rivers (more often between one-fifth and one-half), that which does reach them has ordinarily been so delayed in its progress that the seriousness of the situation is small compared to what it would be without the service of these allies of the welfare of the valleys.

The fact that the general mean run-off of rivers for all seasons and all localities probably averages between 30 and 40 percent of the rainfall upon the watershed must not lead to the impression that this proportion is at all regular. On the contrary, conditions are usually quite the reverse. Streams of decidedly different topographic, geologic and climatic occurrence may vary enormously in percentage of run-off; as some rivers in arid regions whose waters disappear before reaching the ocean and so have no run-off at all, contrasted with those of the other extreme whose flow occasionally approaches 100 percent of the rainfall. It is also true that there is for nearly all rivers a great variation in their percentage from week to week throughout the year, as well as a considerable one for different years. Frequently the fluctuations in yearly averages for a stream are 50 percent from the mean values; and as for monthly records, it is not rare to find the minimum run-off less than one-tenth the average, or the maximum equaling or even exceeding the rainfall of the month. These general facts impressively indicate the necessity for extended and definite observations upon each stream before the planning of engineering works whose usefulness is dependent upon its greatest fluctuating volume of flow.

The three principal causes which result in a usual run-off of only a fractional part of the rainfall are evaporation, absorption by the ground, and the demands of growing vegetation.

Evaporation includes that from the surface of the soil and of streams and lakes, as well as directly from the wet surfaces of vegetation, etc. The amount of all these factors varies much with the temperature and humidity of the air, the temperature of the water or soil surface, the altitude, the prevalence and velocity of winds, and other less important conditions. Because of the great variability

of these contributing factors, the actual amount of evaporation differs very much from day to day, week to week and season to season. It even varies greatly with the character of the evaporating surface and of the locality under consideration. Observed annual averages, for this country, of evaporation from free water surfaces range from about 40 in. in the northeastern states to 100 in. and more in parts of the semi-arid southwest; while the monthly maximum is often five to ten times the monthly minimum. The evaporation from moist soil seems to ordinarily range from about one-fifth for sand to nearly as much as that from water surfaces in the case of loamy earth; but when the top layer is saturated, the evaporation is several times as great as when only moist, while the shade and wind protection of trees or brush and the effect of grass or a surface mulch notably decrease the evaporation from soil.

Transpiration losses occur wherever growing plant life exists, and thus tend to offset the reduction in evaporation produced by their protective influence as just mentioned; losses from this cause are also extremely variable both because of the fluctuating influences already mentioned and the different sorts and densities of the vegetation concerned. It is stated that a single large tree on a very hot day may transpire nearly a thousand pounds of water; and estimates of annual losses of this kind in heavy forests reach an equivalent of about a foot of rainfall, while for grain fields and grass lands they seem to vary from about one-third to an equal amount or more. Compared to these quantities, the amount of water remaining in the growing vegetable fiber is very small. Differently expressed, it has been found experimentally by Wollny and Hellriegel that it requires an average of more than 400 lb. of water to mature 1 lb. of the dry vegetable product, King found the amount slightly greater for Wisconsin, while Widtsoe and Merrill's experiments in the semi-arid region of this country indicate that the average required is perhaps double that just given; and when this process occurs on a clay soil, several times as much is demanded Lafosse estimates that a forest transpires more than 2000 cu. ft. of water per acre each day of the vegetative season. The resulting loss to the soil has been very clearly evidenced by the greater depth of the ground water level in forests, which French and Russian investigators have found to be from 18 in. to 50 ft. deeper than in adjacent plains; the larger figures occurring in the drier climates.

Absorption of rainfall by the ground is also an exceedingly variable quantity, its amount depending upon many factors but especially upon the character of the soil, the nature of its covering and its condition with regard to degree of saturation and temperature at the surface. Of course the ultimate measure of capacity of earth to absorb moisture is its degree of porosity; and even solid rock has a limited capacity to absorb. U. S. Water Supply and Irrigation Paper No. 67 gives the porosity values of various soils and rocks, varying from about one-fourth of 1 percent for the densest granite to 40 or 50 percent for clay loams; it assumes that the average porosity of the 6 miles in depth of permeated materials of the earth's crust is 10 percent, and concludes therefrom that the underground waters are sufficient in quantity to "cover the entire earth's surface to a uniform depth of from 3000 to 3500 ft." On the other hand, Salisbury's "Physiography" states that such estimates have varied from an equivalent surface depth of 100 to 3000 ft., concluding that it "would probably make a layer not more than 1000 feet deep if it were spread out over the surface of the land." In any case it is evident that underground waters form sub-surface reservoirs of enormous capacity which are fed by absorption at the surface of the ground whenever the interstices of the soil are not filled by moisture, and which gradually contribute of their store to various phenomena. This enormous but locally exceedingly variable capacity of soil to directly imbibe a considerably portion of the rainfall is especially affected by its surface condition; for example, a covering of grass will reduce its absorptive capacity to perhaps from one-fifth to one-thirtieth that which would occur if the surface were bare; and with a saturated or frozen surface its absorptive power is gone. Even when averaging the nature and surface conditions of soil throughout the year in regions of ordinary rainfall, the percentage of absorption often varies from twenty to sixty, or more. Typically, the water will pass into the soil until its rate of capacity to absorb is exceeded by the rate of the rainfall, when the excess flows away on the surface to be partly lost by evaporation or the demands of growing vegetation, partly to be absorbed by unsaturated soil encountered in its course, and the remainder contributes directly to the volume of the streams.

15. The Retardation of Run-off by Natural Agencies.—Fluctuation in the quantity of flow of streams depends not only upon the irregularities in the occurrence and amount of precipitation and upon the great variability in the losses just considered, but it is also pro-

foundly affected by retarding influences that exist upon their watersheds. The principal agencies of this kind result from the topographic and geologic character of the drainage basin, the former consisting especially of the slope of the surface and the latter of the absorptive capacity of the soil, while the nature of the soil covering also influences the retarding effect. Although this last factor is usually of subordinate importance, it is of quite general occurrence. Forest lands have a surface composed of leaf litter and humus which is capable of readily absorbing several times its own weight of water; and unforested areas are naturally covered with grass and other herbage whose growth not only restrains the movement of the surface film toward the streams, but whose interlacing roots and matted tops tenaciously oppose the formation of gullies into which this surface sheet of water tends to concentrate and thus expedite its course to the river; the decaying vegetation forms a porous soil whose retarding influence is also beneficial.

The supreme agency of retardation of flow toward the stream is, however, the porosity of the earth. The practically universal occurrence of soil, its enormous extent and great depth, its capacity for absorption and the slowness of flow through it, all combine to give it general preëminence over any and all other factors contributing to this very beneficial result. It is even true that the rocks, notably limestones and sandstones, are considerably porous, and add no insignificant amount to the vast sub-surface water-storage capacity. One has but to recall springs, gushing from rock-fissures in the hills, to be reminded of plain evidence of this characteristic. The great Kaiserbrunnen, furnishing a large part of the water supply of Vienna, bursts from a dolomitic limestone formation about 60 miles from the Austrian capital.

Of that portion of the rainfall which is absorbed by the soil a part is, as before, lost by evaporation and the needs of plant life; a part continues to flow slowly through the interstices of the soil; while a part reappears at the surface, at lower levels than its origin, sometimes as springs but more often in the less spectacular form of moisture exuding imperceptibly from considerable areas. There exist, then, these vast underground streams which are formed by the absorption of water, principally rainfall; and which, in turn, give largely of their store to wells, marshes or rivers, wherever artificial or natural depressions penetrate below their water-table in a soil or rock of sufficient porosity to overcome the retentive influence of capillarity. This universal sub-surface flow conforms to hydraulic

laws; although, unlike the law of surface flow which is fundamentally expressed as varying in velocity in direct proportion to the square roots of the surface slope and the wetted perimeter, the underground flow has been proved by Darcy, Hazen and other experimenters to vary in direct proportion to the first power of the slope of the water-table, the effect of wetted perimeter of course disappearing. The rate of movement varies usually between 2 and 10 ft. per day, though in more porous soils its velocity is sometimes ten times as great. The direction of flow is, of course, determined by the direction of the slope of the water-table which, in temperate climates of average rainfall, is usually from 5 to 20 ft. below the surface of the ground; but it is often deeper; and it coincides with the ground-surface in marshes and swamps, and with the water-surface at the margin of rivers and lakes. In seasons of slight rainfall the greater part, and not infrequently all, of the low water flow of streams is derived from the seepage from this vast underground storage; and the amount of ground water moving down the valley is often a considerable fraction of that flowing in the surface stream, and it may sometimes equal or exceed the visible flow.

The other very notable influence upon the retardation of precipitation in its progress to the rivers is slight surface slope, the effect of which becomes very pronounced in those occasional cases in which it is so small that there results the formation of marshes, swamps or lakes. A very striking illustration of the effect of difference in general surface slope is furnished by a comparison of the relative extreme fluctuation of flow of the Monongahela and the Mississippi River above the mouth of the Crow Wing. Both basins are in a temperate climate, constituting headwater catchment areas whose flow finally reaches the same river. The areas of each are practically identical, that of the Monongahela being 7340 square miles and that of the extreme upper end of the Mississippi, 7283 square miles; the rainfall of the latter is about two-thirds that of the former, it is more generally wooded, and the soil is more porous to a greater depth; but the principal difference in nature of the two basins is that of general surface slope with its accompanying characteristics of a typically swampy and marshy country on the flat Mississippi watershed considered, while that of the Monongahela is hilly and practically lacking in swamps. The slope of the former averages about 1 ft. per mile, and the ratio of its maximum to minimum run-off is approximately six; while the mean slope of the latter averages nearly 14 ft., and the corresponding ratio of fluctuation in flow is about thirteen hun-

dred. While this notable difference is due to a combination of various causes, there is no doubt that the controlling reason is that of the marked contrast in the topographic nature of the two illustrative areas. A similar equalizing tendency upon the flow of rivers has long been observed in cases where lakes exist, their effectiveness for this purpose being the more pronounced as their areas occupy a greater proportion of the total watershed. It has been repeatedly noted that those tributaries of the River Po which flow through lakes are more constant in volume below them than are other branches which lack these partial regulators. In the notable flood of 1856 the maximum volume of flow from Lake Geneva was only 11,400 cu. ft. per second, although its watershed contributed to it at a rate almost five times as great. The very large lakes in the upper part of the valley of the Nile, together with the immense papyrus swamp region near the equator, are said to modify the flow of the river to the north of the Sudd so that its variableness is small. In hundreds of instances in different parts of the world, natural bodies of water are affording similar beneficial results in modifying the great changes that would otherwise occur in the discharge of many rivers. Probably the maximum effect of this kind is produced by our own Great Lakes above the Niagara River, whose areas cover 33 percent of the watershed and which equalize the flow of the Niagara to such an extent that its maximum discharge is less than one and one-half times its minimum. Table Number 1, indicating similar proportions for several rivers, illustrates the fact that ratios

TABLE NO. 1

River	Place of gauging	Discharge in cubic feet per second			Ratio of L. W. to H.W. disch., 1 to	H. W. above L. W.
		Mean	L. W.	H. W.		
Seine	Paris	....	1,250	83,500	67	28 ft.
Rhone	Arles	44,000	17,500	275,000	16	18 ft.
Rhine	Pannerden ..	76,000	32,000	350,000	11	25 ft.
Weser	Bremen	10,500	5,300	140,000	26	..
Danube	Isakcha	207,000	70,000	1,000,000	14	..
Niagara	Buffalo	212,200	168,700	257,800	1½	4 ft.
Merrimac ...	Lawrence ..	7,500	2,000	74,000	37	26 ft.
Ohio........	Pittsburgh ..	....	1,200	434,000	362	36 ft.
Ohio........	Cincinnati ..	....	6,500	676,000	104	73 ft.
Missouri	Kansas City	71,500	10,000	750,000	75	38 ft.
Mississippi ..	St. Paul	....	3,000	117,000	39	18 ft.
Mississippi ..	St. Louis ...	185,000	35,000	1,250,000	36	44 ft.
Mississippi ..	Vicksburg ..	540,000	65,000	2,300,000+	35+	59 ft.

of thirty or more are very common on large ones, such values being typically greater where the stream is smaller in size.

16. Other Complicating Factors in the Control of Stream Flow. —The great fluctuations in volume and velocity of flow and of height of water surface, which are manifested by most rivers, constitute serious difficulties and dangers to a variety of interests of which water supplies for municipal, irrigation, and power purposes, the sewerage of cities and the drainage of wet areas, the protection of life and property from disastrous floods, and the interests of navigation constitute the most important. Although the general advantage of all is found in minimizing such variations, yet the specific concern of each, and especially the direct way of attaining such particular relief for one interest only, is often so limited in character that the others suffer. For example, the ideal situation for navigation purposes is a maximum low water volume, that for irrigation is a maximum during the irrigation season, that for power and municipal purposes is a maximum constant value, the ideal for drainage is a minimum stage, and the essential need for flood protection is a minimizing of the flood discharges only. The lack of concordance in normal methods of attaining such results is illustrated by the fact that especially important requirements in guarding against destructive floods are the prevention of any narrowing of the river by embankments or otherwise, the minimizing of bridge piers and similar obstructions to flow in the waterway, and the straightening and deepening of the stream; while on the contrary navigation interests are, in general, aided instead of injured by a narrowing of a river, bridge piers are not usually objectionable, and a straightening of the stream would ordinarily be a serious detriment to its navigability. These illustrative facts are cited in order to indicate the need of treating rivers with due regard to all the interests concerned in their control, and thus choosing the particular details of improvement to not only accord with the physical condition of each particular stream, but also to secure its improvement in a way to obtain the greatest general advantage. Inasmuch as under present federal laws navigation interests are not only paramount but are practically the exclusive concern of river control, it is evident that other interests may inadvertently suffer. In order to consider and correlate all the different needs involved so that the greatest aggregate good shall follow, it is essential that all the interests concerned should be united under a single control.

The modifying influence of man's activities upon the flow of streams

is generally very slight. This is true in connection with drainage for agricultural and other purposes which, although becoming extensive in certain localities and so at times reducing the retarding effect of ground storage in those places, is yet largely neutralized and even sometimes exceeded by their effect of preventing the saturation of the upper layers, thus permitting rain to penetrate instead of running off the surface and also reducing evaporation. The volume of land so affected is usually small in proportion to the total storage capacity of the soil; so that, all things considered, the modifying effect of drainage averages a comparatively small amount, though sometimes it is locally quite considerable. The same thing may be said of the influence of tillage, especially if proper methods are used. The fundamental agricultural need of so cultivating the fields that surface wash is minimized, in order to conserve the rich soil from being carried into the streams and so wasted, will guard the rivers from the contribution of silt which so complicates their navigability. This procedure will also encourage the absorption of the precipitation by the ground and so will largely neutralize the otherwise unavoidable tendency of tillage to increase the fluctuations of stream flow. With regard to evaporation, although the loss from this cause is comparatively great from cultivated fields, the well-known principle of successful agriculture which requires a thorough and frequent cultivation in order to conserve the moisture will also minimize this unfavorable tendency. While definite conclusions with regard to the aggregate effect are therefore difficult, it is probable that the average adverse influence upon the fluctuation of streams is small; although some writers have considered this effect to be considerable, estimates even running sometimes as high as a 50 percent increase.

In occasional cases the retardation of flow through absorption by the ground, to produce an amelioration of flood and low water extremes, may be obtained indirectly by irrigation. In the semi-arid regions the rainfall is insufficient to saturate the very porous soil, a condition which the irrigation of it, for agricultural purposes gradually produces. Experience is yet far too limited to indicate definite results; yet it is pertinent to mention the fact that the low water flow of some rivers which have been investigated has been steadily increasing in volume for many years through the effect of seepage to them from the gradually saturating irrigated lands of the valleys. It is a quite frequent experience to find the minimum flow of such streams augmented several cubic feet per second per mile from this

cause. For example, comparative measurements reported in "Bulletin 33 of the Experiment Station of Colorado" (1896) indicate that "the seepage from 1000 acres of irrigated land on the Poudre River gave 1 cu. ft. per second constant flow; on the Upper Platte, 1 ft. to about 430 acres; on the Lower Platte, 1 ft. to 250 acres. On the Poudre River about 30 percent of the water applied in irrigation returned to the river. There is a real increase in the volume of streams as they pass through irrigated sections." While the indirect benefit of irrigation is of slow attainment and requires large operations to produce material results, yet whatever influence is produced undoubtedly does promote the desired equalization of flow unless the water needed for irrigation is taken when the river is at low stage; and this benefit will reach its maximum effect when it happens that the irrigation season occurs at flood stage of the stream.

The very considerable number of agencies affecting the run-off and the very great variability of each factor caused by topographic, geologic, climatic, vegetative, annual and seasonal, and other fluctuations and differences, unite to make it impossible to deductively evaluate the influence of each agency and then to combine the individual effects into a result which shall satisfactorily represent the discharge, because of the very great uncertainty of the resulting values. When reliable quantities are necessary as the basis for important determinations, the only adequate procedure is recourse to gaugings of sufficient range and accuracy to satisfy the requirements. When discharge measurements are deficient in extent or otherwise defective, the lack may be more or less approximately supplied by estimate based on the preceding considerations, but controlled in their values by their relation to such gaugings as may be available. In case that a preliminary general estimate is needed, and gaugings have not been made, an appraisement necessarily subject to large possible error is provisionally made from a discriminating consideration of the phenomena already discussed; the usual procedure being the use of empirical formulæ of comparatively local applicability, endeavoring to adapt them to the particular stream in question by such modification as its particular physical characteristics seem to require.

17. The Character and Amount of the Forest Influence.—While it is generally considered that agricultural activities usually have a comparatively small influence upon the run-off of rivers, it is thought by many that forests have a great beneficial effect upon stream flow.

A careful consideration of their influence upon the high and low water extremes of rivers as well as other important effects, such as that of the prevention of the erosion and wash of soil into streams, is very important. Whatever the result may be, it should be noted that the great interests of true forest conservation and reforestation are very real economic necessities, and the justification of a scientifically developed forestry policy is not dependent upon the assumption that it has a notably beneficial effect upon the regimen of rivers. The principal advantageous influences of forests are classified in the five following paragraphs.

1. Observations which have been made to determine the effect of forests upon rainfall indicate that they tend to equalize and increase it locally because of the less variable and lower temperatures in their proximity, which favor the condensation of the moisture of the air. French investigations of long duration indicate an increase of precipitation above forests in amounts up to 30 percent more than that outside; others made in Switzerland, Germany, Austria and other parts of the world have confirmed this tendency, though usually finding a smaller increase in amount of forest precipitation, the excess being usually between 1 and 25 percent; while occasionally the results indicated a smaller amount in the forests.[1] This general influence is greater in mountainous regions than in the adjacent lowlands; and it seems to be usually very small, except perhaps in cases where evaporation from extensive forests is greater than it would be without their presence, thus increasing the supply of atmospheric humidity required for the rainfall in the regions farther inland.[2] The equalizing effect upon rainfall is frequently apparent, although exceptions are numerous. Both these results are generally beneficial, though not necessarily so; as when the resulting increase comes in the spring to augment the flood flow from melting snows.

2. The ordinarily very considerable loss of moisture by direct evaporation from the soil is reduced by forests to a comparatively small amount. This is due to several influences, such as the lower temperature of the air and earth in the warmer weather, the greater humidity, the defense of the trees against winds and sun, and the protection afforded by the humus and leaf litter covering the ground. There are, however, various accompanying factors which unavoidably lessen these advantages to a greater or less extent, sometimes

[1] Senate Document 469, 62nd Congress, 2d Session.

[2] Science, Vol. 38, pp. 63–75.

cause. For example, comparative measurements reported in "Bulletin 33 of the Experiment Station of Colorado" (1896) indicate that "the seepage from 1000 acres of irrigated land on the Poudre River gave 1 cu. ft. per second constant flow; on the Upper Platte, 1 ft. to about 430 acres; on the Lower Platte, 1 ft. to 250 acres. On the Poudre River about 30 percent of the water applied in irrigation returned to the river. There is a real increase in the volume of streams as they pass through irrigated sections." While the indirect benefit of irrigation is of slow attainment and requires large operations to produce material results, yet whatever influence is produced undoubtedly does promote the desired equalization of flow unless the water needed for irrigation is taken when the river is at low stage; and this benefit will reach its maximum effect when it happens that the irrigation season occurs at flood stage of the stream.

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and looseness of earth. Grass performs this service excellently on moderate slopes, and brush and small trees on steeper ones, as in many parts of the Rocky Mountains, and in Wisconsin and other states where there are numerous areas which have lost their original stand of trees but which are now covered with a much more dense growth of small trees that better protects the soil of the hills than was the case before; but when such substitution does not occur, when the slopes are very steep, and where the surface flow is concentrated into gullies, the forests offer an unequaled defense against erosion, particularly because of the extent and tenacity of the interlacing roots of the woodland growths. Perhaps the greatest economic loss from the washing away of the soil has been the transformation of productive territory into barren wastes; and historic examples of the impoverishment of extensive areas are furnished by certain districts of France, Italy, Greece, Palestine and China. The effect which is directly most detrimental to the interests of navigation is the increase of the shoaling of rivers, to which the detritus, originating from the soil wash, later contributes. Such shoaling is often charged entirely to erosion resulting from the destruction of forests, while in fact in nearly all streams the sediment of the shoals also originates partly from tilled fields and partly from the erosion and caving of the river banks themselves. Sometimes one agency predominates and sometimes another; and generally, when a stream is large enough to be navigable, the contribution of detritus and silt chargeable to forest destruction is a very small fraction of the total amount concerned.

The preceding outline indicates that the average influence of forests upon stream flow is usually beneficial, particularly in lessening their excessive fluctuations in volume and in reducing their load of sediment, although the opposite effect is sometimes produced. It also reveals the complexity of the situation, gives evidence of the fact that no two cases can be alike in the resulting measure of benefit, and illustrates the present hopelessness of attempting to deductively determine the degree of damage to be expected from the cutting of forests or of benefit to result from reforestation. The amount of their beneficial influence is desired in order to determine whether this factor, which is one of the few that is at all controllable by human agency, may be practically available to alleviate adverse conditions. The real proposition is necessarily based, in each case, upon economic grounds; and this involves direct comparison of the total cost (including a greater or less resulting restriction of agricultural operations and development) with the measure of advantage

secured by the alternative forest influence. The qualitative consideration already discussed must therefore be supplemented by a quantitative investigation in order to adequately estimate the actual value of the effect of forests upon rivers.

Unfortunately for definiteness of conclusions, the determination of the amount of advantage is also a most complex question; for its satisfactory investigation would require either two comparative areas of identical characteristics in every respect except that of forest cover, which should be as unlike as possible; or else a series of thorough observations in the same watershed when forested, and again when the forests have been largely removed. The former method is subject to the criticism that no two areas are alike in soil, rainfall, temperature, slope and the many other variables except the chosen one of forest condition; and the latter, that at no two periods is the same area visited with a recurrence of climatic and other influences so precisely similar that any difference in volume of flow could be ascribed unqualifiedly to differences in forest condition. Hence, such investigations must be made with scrupulous care in order to justly receive credence; and even then the conclusions will be approximate, not exact.

It is surely unwarranted that many should assume, because of the proposition that forests ordinarily exert a beneficial influence, that this effect is necessarily of such magnitude as to be generally of marked importance in the phenomena of rainfall, mean discharge, floods and low waters. Scores of instances have been cited which seem to indicate a direct relation, but usually such statements lack the evidence of that keen analysis which is necessary to determine what part of the total effect is due to the combination of causes other than forest influence, and so to reduce the argument to its ascribed terms. For example, it is no proof of the existence or non-existence of such relation to state that, because the rainfall of a state averaged a small fraction of an inch less for ten years after extensive timber cutting than it did for the twenty-five years before, therefore deforestation reduces rainfall; it is a fact disclosed by records that precipitation averages, for considerable periods, are sometimes high and sometimes low, with periodic recurrences of fluctuations which could superficially be made to corroborate any desired theory. Nor can the bare facts, that the greatest flood known in the Mississippi at St. Louis occurred in 1844 while the primeval forests were practically untouched, or that as numerous and as great floods devastated this river a half century ago as has been the case since the extensive

clearing of timber from its watershed, be considered convincing evidence of the opposite kind; because such conditions result from a combination of causes and the influence of the one is lost among that of the others. A similar criticism applies very definitely to all hastily assumed or superficially adopted conclusions which have been so often seized upon by an eager partisan and reiterated by one and another until the average individual thoroughly believes that the question is settled with an exactitude which is, in fact, Utopian. Even the extensive and careful studies of the last century of such European rivers as the Seine, Loire, Po, Rhine, Elbe, Danube and Volga, offered indications which permitted very enthusiastic claims for those advocating the great importance of forests to rivers; and yet at the same time actually, by different interpretation of the same data, gave opportunity for strong denial that such influence was definitely shown.

Thus the question still needs investigation and the most discriminating scrutiny. Quantitative results are few, and their definiteness may be expressed as an approximation or a tendency rather than a matter of assigned value. Yet in recent years there are several instances of thorough studies of this kind in this country, the results of which are of much significance.

In Volume I of "Professional Memoirs" (1909) are given the results of the investigations of the flow of the Tennessee and the Cumberland Rivers for a period of about thirty-five years. Both are navigable. The drainage area of the Cumberland above Nashville is 11,600 square miles, and that of the Tennessee above Chattanooga is 21,418 square miles. The estimated area of these watersheds under forest cover at the time of the study is given as about 60 percent, a reduction of about 20 percent during the latter portion of the period under consideration. The cutting of timber occurred principally on the higher parts of the watersheds. The scrutiny of the records and of comparative charts prepared from them are stated to indicate the following facts: (a) the considerable destruction of the forests has had no noticeable effect upon precipitation; (b) while the latter part of the period averaged low in rainfall with a consequent reduction in flood heights and in duration of high waters, yet the low waters were generally higher than usual; (c) no reduction was discoverable in navigability on either river due to the silting up of the streams, conditions now being better than they were thirty years ago; (d) if any adverse effect upon stream flow was caused by cutting of the forests, it was too slight to be detected.

The second important series of extensive investigations of this question was based upon the records of several Wisconsin rivers, the detailed discussion of which is contained in the "Bulletin of the University of Wisconsin, No. 425" (1911). The rainfall and run-off records of the various watersheds considered were available for periods ranging from thirty to sixty years, and the rapid cutting of the forests of this state occurred largely during the latter portion of the period considered. The opinion is set forth that whatever effect the forests have upon the stream flow, which is of practical importance as distinct from theoretical inference, will be evidenced by such change in the phenomena of flow after deforestation has occurred as will be clearly apparent. A careful study of the records, diagrams and charts of the different rivers led to the following general conclusions; (a) any effects which the extensive cutting of timber may have had upon stream flow were obscured by other changes in condition, such as increase of agricultural operations and drainage, the second growth of trees, and brush, etc.; (b) on the Wisconsin River the changed conditions have not influenced the low water and mean flow, but appear to have caused a slightly greater regularity of ratio of precipitation to run-off and also to have increased the flood heights by a small amount, but these expected tendencies were insignificant in amount on this river, and were wholly lacking in the case of the Upper Mississippi and the Wolf rivers; (c) as a general proposition timber cutting has had no material influence upon the regularity of stream flow, nor any particularly adverse or favorable effect upon low water, high water or mean discharge; (d) the study of these rivers discloses no evidence that reforestation would in any way affect the regularity or volume of the flow of the rivers, or reduce the heights of their flood waters.

A very extended and thorough investigation is described in "H. R. Doc. No. 9, 62nd Cong., 1st Session" (1911). The rainfall records of stations scattered about the watershed of the Merrimac River have been available for periods varying from fifty to ninety years; the gauge records at Lawrence give run-off data for a continuous term of sixty years, and the drainage area above this city, where the discharge measurements have long been required for water-power purposes, is 4664 square miles. This area has been subjected to a progressive deforestation from early times to about the year 1870, since which time there has occurred a gradual reforestation which is estimated to amount to about 25 percent at the time of the investigation. This typical watershed is therefore considered to offer

a particularly favorable opportunity for such research, and the conclusions of the investigator, as derived from his diligent analysis of the extensive records available, are those of one who had "a strong predisposition in favor of the popular belief that forests do exert a material beneficial influence upon stream flow." The more significant results may be summarized as follows: (a) deforestation was not accompanied by a reduction in precipitation nor did reforestation produce any increase, the changes in rainfall bearing no relation to the changes in the forested areas; (b) the mean discharge of the river is not appreciably influenced by changes in the forested area of the watershed amounting to more than 25 percent; (c) the cutting of the timber has not reduced the low water stage nor has it lengthened the low water periods, neither has reforestation ameliorated these conditions; (d) the height, the volume, the duration or the frequency of floods have not been increased by deforestation nor mitigated by reforestation.

The fourth instance of such an investigation that may be advantageously cited is one inaugurated by the U. S. Geological Survey in May, 1911, upon two adjoining watersheds of about 5 square miles each, which lie in the White Mountains and whose waters finally reach the Merrimac River whose salient characteristics were considered in the last paragraph. The basins of Shoal Pond Brook and Burnt Brook are described as very similar in all important respects except that the former is wooded to an amount of about 80 percent of its area, but the latter has been practically denuded of its forest cover and subsequently burned over. The amount of precipitation and its distribution over both of these small watersheds are regularly observed with especial refinement, and the run-off from each is also carefully determined at stations as customarily established, equipped with automatic recording gauges. The preliminary report (1912) attaches great significance to the results so far observed, based upon three selected storm periods of April, 1912, of several days each. There was a deep accumulation of snow, with considerable precipitation and thawing during these times considered. A comparison of the two basins showed the following conditions: (a) during all of the three periods there was more snow on the Shoal Pond Brook than on the Burnt Brook watershed; (b) the amount of water held in snow-storage was diminished only about 70 percent as much on the basin retaining its trees as on the other; (c) the run-off during these three periods of April was only half as great from the forested area; (d) the maximum flood flow from the wooded basin was but two-

thirds as great as that from the deforested one. At the close of the last period there was still remaining an average depth of snow of 6 in. on the latter watershed and of 14 in. on the former. Comparative conditions for the intervening and following weeks and at other seasons, as well as for other years, would be interesting; and a final report is promised after a more extended period of study, although none has yet (1914) appeared. However, these results are stated to be such as to "satisfy the most radical forest enthusiast."

The contrast between the last case and those of the first three cited is evident; but the differences are more apparent than conflicting. There is no possible doubt that a forest cover will, under certain conditions, produce as great retarding results on relatively small areas as those given in the last paragraph; under other conditions it will show probable influences of other amounts; and sometimes the influence will be adverse, as was the case in the instance of the Cedar River cited above. Yet the effect on comparatively small areas is undoubtedly definitely beneficial in most instances.

When the watersheds are large in area the situation is different for several reasons, the principal of which are the effects of local precipitation and of increasing lack of synchronism of flood crests of the different streams as the basin increases in size. With the usual limitation in extent of ordinary precipitation it is very evident that a relatively small area is proportionately much more likely to have its whole extent exposed to a rain than is a large area; or the effect of a storm of a certain extent would be only one-tenth as severe on a watershed of ten times that extent as it would be on an area equal to that covered by the rain, other things being equal. Again in the case of general precipitation so extended as to overspread the larger watershed, similar conditions will usually produce a comparatively moderate discharge on the more extensive basin because of the corresponding probability that the duration of the precipitation will not be great enough to produce the full effect upon it.

One of the most notable agencies of moderating the fluctuations in volume of stream flow, which is increasingly effective as the area considered is the greater, is that the flood crests of the different branches of a stream are rarely coincident in the time at which they reach the same point. A brook draining a basin of several square miles of area will be discharging its maximum at a certain time; the creek receiving it is rather unlikely to be charged with the greatest flow of other branches at the same time that the effect of the flood peak from the first brook passes the mouths of the others in succession. As the

creek with its tributary area of scores of square miles unites with other creeks to form a small river, the probability of a synchronism in their flood crests is still less; and as successive rivers draining hundreds of square miles unite to form main streams, such a continued coincidence in the various flood volumes reaching successive sections of the large river at the same time is almost impossible. It is this usual diffusion of effect of floods in successive affluents of a river, which is generally characteristic, that powerfully contributes to reduce the large contrasts in run-off of restricted areas to the small, and even undetectable, differences in large rivers resulting from differences in forest conditions.

The characteristics of regimen most important for consideration as usually given in connection with forest effects are rainfall, average run-off, flood and low water flow. As for the effect on precipitation, the Geological Survey investigation already considered states that "it is not contended here that deforestation changes rainfall occurrences"; and concerning the relation between mean annual rainfall and mean annual discharge the statement is made that "there is no well established claim that an alteration of these relations is caused by a change in forest cover." There remains the question of flood and low water effects, and it now seems evident that these exist in extremely varying amounts, but typically considerable in the streams of small watersheds, and relatively small in rivers draining large areas; so slight, indeed, because of the various equalizing influences which have opportunity to occur on great river basins, that they may generally become imperceptible. It should be noted that the most important of these four effects upon their navigability is the low water condition; and it is reasonable to infer that the relatively large run-off claimed for deforested areas, in the case of summer and fall showers, when rivers are usually low, would be especially advantageous to navigation, rather than the reverse.

18. The Relation of Storage Reservoirs to the Control of Run-off. —It thus appearing that the influence of forests upon stream flow is of material consequence only on the smaller streams, attention is directed to the two other extensively employed agencies of human control; reservoirs offering the most notable artificial method of regulating run-off and levees forming the usual system of defense against floods. The latter will be considered in Chapter IX. With regard to storage reservoirs it may be said at once that, although entrained sediment and suspended silt entering them in the tributary streams is deposited on reaching their quiet waters, yet the con-

sequent reduction in capacity is ordinarily so small as not to be important, and often it is relatively negligible. Yet reservoirs, in common with natural lakes, swamps, etc., do perform in this regard a service to the interests of navigation of the river below in thus causing the removal of such detritus which would otherwise reach the waterway. The Rhone is very turbid as it enters Lake Geneva, but is clear as it flows out; the same condition characterizes the Rhine from Lake Constance, the Neva from Lake Ladoga, the St. Lawrence from Lake Ontario, and practically all similar cases.

The very function of reservoirs, that of filling, holding and emptying their supply of water as required, abstractly renders them particularly adapted to serve to reduce the natural fluctuations in volume of rivers. This influence is similar in kind to that of lakes as already discussed, but is theoretically even more effective because the release of the waters of a reservoir is wholly under control while that of natural lakes is relatively unrestrained. Thus reservoirs seem to offer an ideal opportunity to mitigate the misfortunes incident to the high and low water extremes of rivers. But there are two practical considerations which limit their employment; these are the storage capacity required for effectiveness, and the ever present question of cost.

Impounding water to supply the needs of municipalities, irrigation, power production, navigation canals, etc., has been the practice of different nations of the world for centuries; but a slight consideration will indicate the great contrast between the direct utilization of a volume of water as a commodity for such a purpose, and the meager effect on the height or discharge of a flood produced by abstracting that volume from the total quantity, or by adding the same volume to the low water flow of a stream to increase its discharge and depth as it passes. For example, a billion cubic feet of stored water will supply a city of one or two hundred thousand inhabitants for a year; or it will irrigate from 4000 to 10,000 acres of arid land for a season; or it will furnish more than a million horse-power-hours under a head of 40 ft.; but it would double the volume of low water flow of the Mississippi River at St. Louis for less than eight hours, and is exceeded by the flood discharge at the same place in one-quarter of an hour. Another illustration is that of the vast storage capacity required for an approximate equalization of flow as exemplified by the Great Lakes above the Niagara River, whose remarkable steadiness is secured by their natural storage capacity of some eight thousand billions of cubic feet existing between the high water

and low water surfaces; even the cubic contents of the waterway of the Mississippi River from the Ohio to the Gulf from the low water level to the flood plane, is about two thousand billions of cubic feet.

The relatively enormous volumes of water, which are involved in the approximate equalization of flow of navigable streams are thus usually found to be entirely impracticable to manage, and such endeavor is therefore reduced to the consideration of a possible storage capacity that will serve to prevent the destructiveness of floods and to relieve the limitations of low water; and even this relatively moderate amelioration is often prohibitively extensive, and only occasionally is found to be justified by existing conditions. We have only to recall the vastness of the natural agencies, whose interposition holds back much of the excess rainfall to later contribute it gradually at lower stages of the stream, to conclude that any human aid to this regulation must also be on a large scale if it is to produce perceptible effects.

In Europe, some notable instances of extended investigations of the feasibility of reservoirs to prevent disastrous floods are those made by the French government, following the great floods of 1856, and again after those of 1875. In discussing the principal reasons for finally abandoning what appeared to promise relief for the Seine, Rhone, Garonne, Loire and other rivers,[1] reference is made to the very great capacity and expense involved in adequate reservoir protection, and to the complications resulting from their gradual filling with sediment; to the origin of some floods from heavy rains occurring in the lower part of the river valley, and therefore beyond the influence of the reservoirs, as was the case on the Garonne in 1875; and to the great difficulty which at times would be involved in their operation so that they will always be empty on the approach of flood conditions (there were five in succession on the Garonne in 1856), and so that the filling and emptying of reservoirs on different affluents should never result in unfortunate coincidence of high water discharges in the main river. Those projects were therefore relinquished because of the great capacity required, their excessive expense, and the impossibility of planning the systems of reservoirs for adequate protection against the different combinations of rainfall and run-off conditions which might occur on the different tributaries of the various rivers. The most important rivers of Hungary have been similarly the subject of careful investigation.[2] While appre-

[1] Annales des Ponts et Chausées, 1881, Vol. 2, p. 5 et seq.

[2] Brochure 8 of International Congress of Navigation, 1912.

ciating the importance of reservoirs to the interests of navigation, the conclusion is reached that they are only practicable when also advantageous for irrigation and power production, because of their great cost. For example, reservoirs to furnish the 3,000,000,000 cu. ft. required to augment the low water flow of the Bega River would have cost more than \$1600 per million cubic feet. In 1893 a similar solution of the low water amelioration of the Bohemian Elbe and Moldau was rejected for the same general reasons. However, there are reservoirs primarily for flood protection, but also affording power development, on headwaters of the Bohemian Elbe and Oder Rivers, whose total capacity exceeds 800,000,000 cu. ft. and whose cost averaged about \$4400 per million cubic feet. Probably the storage reservoir of the Wien River, built with a capacity of more than 500,000,000 cu. ft. to protect the city of Vienna from flood damage by that torrential stream, indicates the extreme of unit cost; it was almost \$30,000 per million cubic feet. In Germany a great development of storage reservoirs for power development, flood protection and the improvement of navigation has occurred. One on a tributary of the upper Weser River has a capacity of 7,147,000,000 cu. ft. which is to be utilized to increase the low water flow, develop power and mitigate the floods; the cost is about \$660 per million cubic feet, and the especial benefit to navigation is the increase of minimum depth of the Weser from about a foot in the upper portion to about half that amount in the lower part of the 200 miles below Münden. For the relief of low water deficiency and flood protection on the Oder River, two reservoirs are planned with a combined capacity of 6,730,000,000 cu. ft., the cost of which is expected to be about \$1000 per million cubic feet; it is the expectation that the added flow from these reservoirs will furnish the increased depth of about three-fourths of a foot maximum required for navigation below Breslau, and so make unnecessary the greater expense of extending the works of canalization of the upper river to this middle portion. The thirteen reservoirs of this river basin whose purpose is mainly flood protection, but also the production of power, aggregate a storage capacity of 3,340,000,000 cu. ft., and their cost averages \$1340 per million cubic feet. On the Rhine watershed there are storage reservoirs whose combined capacity is about 8,500,000,000 cu. ft., costing in the vicinity of \$1500 per million cubic feet; their purpose is flood protection, the development of power, etc.

There have been a considerable number of extensive investigations in the United States of the feasibility of a partial control of stream

flow by means of reservoirs. Nearly thirty years ago a series of forty-one dams in Minnesota and Wisconsin were considered in connection with the improvement of navigation on the upper Mississippi River, but the conclusion was reached that this could be more advantageously and economically secured directly by works of improvement in the river channel itself rather than by the impounding of water on the St. Croix, Chippewa, and Wisconsin Rivers.[1] The disastrous floods of the Kaw River, especially at Kansas City, led to an investigation of the possible efficacy of storage reservoirs; but the initial cost, estimated at $11,000,000, and the annual loss of nearly $600,000, due to lands necessarily withdrawn from occupancy, led to an adverse report.[2] The relief to Kansas City was later secured by a systematic widening and deepening of the channel and freeing it as much as possible from the encroachments of bridges and other obstructions, and by the building of protecting levees. A proposal that the flood conditions and the low water navigation of the Red and Minnesota Rivers be improved by the conversion of several lakes upon their headwaters into storage reservoirs was adversely reported upon because the measure of advantage that would result was considered insufficient to justify the cost.[3] A comprehensive system of storage reservoirs on the headwaters of the Missouri River has been proposed,[4] and five of the more important series of sites have been reported upon. The total storage capacity of those so far considered is nearly 14,000,000,000 cu. ft., and the estimated cost is about $125 per million cubic feet. The construction of these and other reservoirs was strongly recommended because of their great value for power and irrigation purposes, especially the latter; the advantage from the effect upon floods and the increase of low water flow of the navigable rivers was considered too small to alone warrant the expenditure.

Probably the most ambitious proposal of the sort yet suggested concerns the headwaters of the various branches of the Ohio River in the Appalachian Mountains.[5] Summarized, the tentative plan involves altogether one hundred storage reservoirs, the expected

[1] Report, Chief of Engineers, U. S. A., 1887, pp. 1681–93.

[2] H. M. Chittenden's "Forests and Reservoirs in their Relation to Stream Flow with Particular Reference to Navigable Rivers," in Transactions American Society of Civil Engineers, Vol. 62, p. 295.

[3] Report, Chief of Engineers, U. S. A., 1904, pp. 2260–2296.

[4] Report, Chief of Engineers, U. S. A., 1898, pp. 2816–2886.

[5] From Preliminary Report of Inland Waterways Commission, pp. 451–90 (1908).

capacity of which totals more than two thousand billions of cubic feet. The estimated cost of these proposed reservoirs is given as averaging about $62 per million cubic feet, though apparently this does not include the expense of lands required, etc. It was claimed that the given amount of storage would reduce the floods of the Ohio Valley practically to or below the danger line, would make unnecessary the construction of many of the projected dams and locks of the canalized river through the increased volume of flow thus available, and would make possible a great and valuable development of water power. Concerning the estimated cost of these reservoirs, it may be said that there is always a tendency for a stated cost to increase rapidly from that of the low preliminary estimate, the moderate appraisement, the more complete determination, to the final actual cost of construction. In this case the cost of construction cannot be given; but successive approximations toward it have since been made possible in the case of the Allegheny and Monongahela River watersheds. Applying the method of the preliminary estimate, the cost of the originally proposed storage capacity on these basins of about 250,000,000,000 cu. ft. amounts to nearly $34,000,000, or about $135 per million cubic feet. Now the investigation of methods of flood protection for the city of Pittsburgh[1] involved a more definite study of storage reservoir sites, and the probable cost of impounding 80,498,000,000 cu. ft. on these watersheds is given as $34,170,800, or $424 per million cubic feet; while the estimate on the seventeen reservoirs as recommended for construction by the Commission is $21,672,100 for a capacity of 59,481,000,000 cu. ft., or $364 per million. More recently the Federal Government required an investigation of this general project to determine if it would be justified in coöperating in the proposed plan because of the resulting improvement to navigation. The report of the Board[2] states that additional sources of information, especially those of private water storage projects whose prosecution has developed more definite values, have enabled it to more closely estimate the cost of sixteen of the proposed reservoirs, the seventeenth being now under construction for power development. The increased land and property damage assigned by this Board amounts to $12,424,110, and this brings the unit cost of the sixteen reservoirs to about $610 per million cubic feet, an amount more than four times the preliminary estimate and about 50 per-cent greater than that of the Flood Commission. With regard to

[1] Report, Flood Commission of Pittsburgh, Penna., 1911.

[2] H. R. Document No. 1289, 62nd Congress, 3rd Session.

federal aid the report was adverse, concluding in general that the benefits to navigation would be so slight that federal expenditures would not be warranted, citing in this connection that where reservoirs are used for the two combined purposes of flood protection and increase of low water flow only a part of their capacity is safely available for each; that contracts already exist for reservoirs for the development of power and many more are planned, and these interests are both important and will exert some favorable tendency upon both the question of flood protection and amelioration of low water stages; that for the slack water system now being constructed on the Ohio River the only advantage would be "a slight prolongation of open water navigation," while the proposal of omitting the locks and dams in consequence of increased low water volume is not possible, as illustrated by the fact that to secure the 9-ft. depth at Wheeling would require a storage of nearly 400,000,000,000 cu. ft.

There are three notable reservoir systems, of unusual capacity, designed particularly for the aid of navigation. In the upland country of central western Russia, about 200 miles southeast of St. Petersburg, where the sluggish surface waters of the marshes and numerous small lakes flowed partly into the Msta and thence northwestward toward the Baltic Sea and partly into the headwaters of the great Volga River, a series of storage reservoirs has been constructed which store about 35,000,000,000 cu. ft. of water for a controlled discharge during the season of low water. The largest reservoir of the series was built in 1843, raising the water surface only 20 ft. above the original low water level at the site of the crib dam; and yet it is 60 miles long and contains more than 40 percent of the total capacity of the system. The 20,000,000,000 cu. ft. available for the Volga is drawn upon for an average period of about three months, and the increase of depth of low water is said to amount to nearly 3 ft. at a distance of 100 miles, half that amount 200 miles downstream, and one-seventh of a foot at a point 400 miles from the reservoirs for a sustained discharge of 2100 cu. ft. per second.

At the headwaters of the Mississippi River, a similar almost level country of swamps and lakes gives another rare chance for impounding water with low dams. Five were constructed of timber about thirty years ago at Lake Winnibigoshish (including Cass Lake), Leech Lake, Pokegama Falls, Sandy Lake and Pine River,[1] at an original cost of about $700,000. The elevation of normal high water surface above that of normal low water in the different reservoirs

[1] Report, Chief of Engineers, U. S. A., 1906, pp. 1443–74.

is from about 6 to 16 ft., and their total storage capacity is 93,237,000,000 cu. ft. After a score of years of service these dams have been rebuilt of concrete, holding the reservoirs to substantially the same levels and dimensions. In 1912 the sixth reservoir, that at Gull Lake, was added to the series, making the aggregate storage capacity of this system 97,846.900,000 cu. ft.; the total cost of which, including renewal, land damages, maintenance, etc., is nearly $18 per million cubic feet. The reservoir surfaces cover about 11 per-cent of the area of the watersheds at high water. While the mean annual run-off of these basins, and consequently the average volume available for increasing the low water flow of the Mississippi, is only about half the total capacity of the reservoirs, yet the storage is valuable to carry over to the years of deficient rainfall (when the run-off is about one-seventh their volume) a part of the flow of years of excessive precipitation (yielding a quantity approaching the storage capacity).

"The reservoirs are operated mainly with a view to the improvement of navigation on the Mississippi River, but with due regard to other legitimate interests. Incidentally they are of great benefit in mitigating floods and in regulating the flow of water for power purposes. No definite schedule can be determined beforehand, but the following are the general rules observed in operation: (a) The discharge must not, by operation of the reservoirs, be reduced below the normal low water flow of the streams affected. This rule is necessary in the interest of manufacturers. (b) When logs arrive in the reservoirs they must be sluiced through. Transportation of logs by floating is a form of commerce and the main form of commerce on the streams affected by the reservoirs. It is dangerous to the dams to allow accumulations of logs, so that they must be sluiced through even in times of flood. (c) The winter flow is so regulated as to make room for 39,000,000,000 cu. ft. of water at the end of winter. This is the amount ordinarily to be expected in the spring floods. (d) From the spring thaw until the dry season of summer (ordinarily until about July 10) as much water is retained in the reservoirs as possible, subject to rules (a) and (b). (e) When the gauge at St. Paul has fallen nearly to 3 ft. (which reading indicates a channel depth at St. Paul of 5 ft.) water is released so as to keep the gauge at this reading. If there is not enough water for this purpose, then the greatest constant depth possible is maintained. (f) When, during the low water stage, there is not sufficient depth for the steamer plying between Aitkin and Grand Rapids and the quantity of water in the reservoirs is sufficient, enough water is released, on request, to make a trip possible. This use of the reservoirs is occasional."[1]

[1] Report, Chief of Engineers, U. S. A., 1910, p. 635.

Three-fourths of the storage is in reservoirs somewhat more than 400 miles above St. Paul, and consequently the maximum effect on low water depth exists here where the river is quite small; however, navigation interests are comparatively slight at present in this portion of the river. At St. Paul the natural extreme low water volume is liable to be as low as 1500 cu. ft. per second, but with the available storage of the reservoirs this can be readily kept at 6000 or more. It is considered that the adopted method of operation of the reservoirs during the low water season in the interest of navigation, as already quoted, will practically result in a maximum increase of depth at St. Paul varying in different years from 5 to 40 in. and averaging 22. This substantial advantage would hold for a distance of 22 miles below St. Paul; but from this point the increase of stage due to the reservoirs will rapidly diminish as successive large tributaries add their volume of flow, until the effect becomes relatively very small at Winona, 125 miles below St. Paul.[1]

A third system of such reservoirs is being formed on the headwaters of the Ottawa River,[2] where the Canadian Government has one concrete dam finished, two are under construction, and contract plans for the fourth are under way. These are designed to raise the level of Lakes Timiskaming, Kipawa, Quinze and Expanse 20 ft. above the present water surface and thus secure a storage capacity estimated at 168,000,000,000 cu. ft. These proposed reservoirs being in the wilderness, the total expense, including land damages, surveys and construction, for securing this vast capacity, is estimated at only about $5 per million cubic feet. It is expected that these regulating works will ultimately increase the low water flow at Ottawa by a volume of 10,000 to 12,000 cu. ft. per second for a low water period of five months. The stated purposes of this reservoir system are to improve the potability of the water, to reduce flood heights, to steady and augment the flow for power production, and especially to increase the low water depth for navigation. It is also considered evident that additional storage capacity will be advantageous, and several more sites are now being studied and others are contemplated.

These three reservoir systems are the principal ones of the world whose use is primarily devoted to the improvement of the navigability of rivers. They are notable for their enormous capacity.

[1] Report, Chief of Engineers, U. S. A., 1887, pp. 1683–93.

[2] Canadian Government "Reports of the Ottawa River Storage," etc., 1912.

They are also unique in their low unit costs. Because of the increment of flow from successive tributaries, the exclusion of the stored ground-water contribution which would be added to the low water flow if the river continued to recede, and the various direct losses occurring, the influence of a reservoir supply reduces so rapidly (as the distance from its place of storage increases) that the quantity impounded must be very great to produce effects that are worth while. The situation is further sometimes complicated by difficulties of operating the reservoirs to the maximum advantage of the low water flow, and by the instability of the bed of alluvial rivers. Examples of the former are pertinent in connection with the fact that released water may be not needed, or may even be harmful when it reaches the intended stretch of river because of local rise there, occurring during the time of flow from the reservoir to the place; for the reservoirs of the Mississippi the time of flow is about twenty-one days for the first 70 miles and sixteeen days for the remaining distance to St. Paul. Illustrations of the latter are furnished by the upper Mississippi River, on which the crests of bars often rise as fast as does the river, up to a 5-ft. stage; or by the lower Mississippi, on which the gain in depth is typically only about half the increase of stage.[1] Consequently the unit cos must be very low in order to justify the impounding of water to aid open river navigation; unless special conditions exist, such as avoiding the necessity for expensive canalization if the deficiency in flow can be thus supplied,[2] or unless other interests are also served.

The usual cost of storage reservoirs is measured by hundreds or thousands of dollars per million cubic feet. To vindicate such an expenditure, it is necessary that they perform some particularly valuable service. Notable illustrations of recent construction for the more usual purposes are the Ashokan reservoir of the New York City water supply, with a capacity of 17,650,000,000 cu. ft., whose contract price amounted to $718 per million cubic feet;[3] the Assouan reservoir of the River Nile, whose capacity of 81,200,000,000 cu. ft. is impounded at a capital cost of $300 per million cubic feet, for the irrigation of vast tracts in Egypt;[4] the Keokuk reservoir of the Mississippi for power production; and the reservoirs of the Wien for the protection of Vienna from floods, as already

[1] Final Report, National Waterways Commission (1912), p. 193.

[2] Brochure 2, International Congress of Navigation, 1912.

[3] Engineering Record, Vol. 63, p. 414.

[4] Engineering (London), Vol. 95, p. 568.

noted. In fact the great majority of storage reservoirs actually do serve more than one purpose, of which the improvement of navigable conditions of the river is sometimes one; while the development of power is often the controlling utilization, and is now rapidly becoming the predominant one.

In comparison with the importance of improving the low water flow, navigation interests are but slightly concerned with the control of the high water discharge; and therefore it is true that the protection of property is the particular object of flood control. The assistance of storage reservoirs in this purpose may vary from a comparatively small amount available when a flood flow finds the reservoir already full, and so is serviceable only to the slight extent produced by the retarding effect of the expanse of its surface, as is a natural lake, to a complete protection such as exists when the total reservoir capacity is great enough to contain any possible flood excess, and its location is not so far upstream as to impair its effectiveness. Occasionally storage reservoirs constitute the complete defense, as in the case of most of the Silesian tributaries of the Oder River, already referred to; but usually they are auxiliary to, or are replaced by, other methods of protection; such as embankments, as have been constructed on the River Po; or the rectification or enlargement of the river, as on the Danube; or the occasional construction of overflows whose crests are somewhat below the tops of the levees and allow the excess volume to temporarily flow into old channels or other selected lowlands whence they later return to the river farther down stream, as on the Yssel and Maas; or the employment of a relief channel, as proposed to reduce by several feet the high water level at Paris. Sometimes a combination of several of these expedients proves the most effective; and not infrequently the assistance of storage reservoirs is inadvisable, as in the case of the lower Mississippi where lateral reservoirs[1] would submerge an immense area of land of great value for agricultural purposes, which is now being drained and reclaimed; while the reservoirs upon the headwaters as proposed for the watersheds of the Ohio and Missouri, and those already existing on the upper Mississippi, would sometimes be entirely ineffective. This last statement is evident when it is recalled that the record flood of 1912 was due directly to a succession of great storms passing from the Gulf to Missouri and Kentucky and resulting in a precipitation for March of 2 to 4 in. more than the normal over all this great central area; the river stages of the Missouri

[1] Journal, Western Society of Engineers, Vol. 5, pp. 259-320.

at Sioux City and of the upper Mississippi at Rock Island were less than two-thirds that of previous flood records, that of the Ohio at Cincinnati and the Cumberland at Nashville were only about three-fourths the previous record, and that of the Tennessee at Chattanooga was but slightly more than half its maximum high water stage. Again, it is estimated that, if storage reservoirs had been in existence, capable of preventing all flow of the Ohio at Pittsburgh, the Mississippi at St. Paul, and the Missouri at St. Joseph, their combined influence upon the great flood of 1913 would have been to reduce it in height hardly 6 in. at Cairo,[1] and of course a lessening amount in its progress onward to the Gulf.

A reservoir control of floods is a local question. The physical conditions of a river basin, the flow data of the affluent streams, and the relation of each to the other, all unite in fixing the degree of relief from floods which storage may secure. To be effective the water impounded must invariably be that which would have formed a portion of the actual flood volume had it not been withheld; and this is not only impossible in the case of precipitation which occurs between the place of storage and that whose flood relief is sought, and of the contribution from intervening tributaries, but the effectiveness of reservoirs is only partial, always variable, and somewhat indefinite. This is due to the variability of the run-off in different storms from each of the several tributaries above the river section in question, and to the differences in time at which the contribution from each reaches the place of danger. A study of any flood at this place to be protected will develop the hourly volume of flood flow. A detailed investigation, which involves the factors of discharge and length of time of flow from a proposed reservoir site to the place in question, will disclose what portion of each hourly flood volume originates above that reservoir site; and a like determination indicates the similar portions which come from other storage sites on other portions of the headwaters. Some tributaries will thus be found to be contributing more than others during the period of overflow, and these are the more effective during that particular flood. Now, to secure protection from a like flood, it would be necessary to impound at least that quantity of water, hour by hour, in a selected number of the proposed reservoirs, which represents the hourly flood excess at the place which is to be safeguarded. An identical procedure is followed in the case of all other floods; and, of course, different quantities and combinations

[1] Professional Memoirs, Vol. 5, pp. 424-425.

are found to exist in each one. Finally, the studies of these different probable flood conditions are combined, and the situation and capacity of each of the adopted reservoirs must be so fixed that their aggregate capacity and effect shall safely exceed the flood excess at every hour of any of the anticipated storm conditions. It is these relative values for the different storage sites, as well as their practicable storage capacities, which make some reservoir locations more effective than others; and it is the fact that the influence of each proposed reservoir varies in each flood, because the run-offs differ, which makes necessary a total storage capacity greater than any single flood excess. For example, the investigations of the Pittsburgh Flood Commission found that forty-three reservoirs with a total capacity of more than eighty billion cu. ft. would be inadequate to prevent a flood excess at Pittsburgh of less than one-third that volume, reducing the 1907 flood plane from 35.5 to 25.3 ft.; while omitting twenty-six of the least effective of these, and so reducing the proposed storage capacity to less than sixty billion cu. ft. would increase the flood level only 2.3 ft., and would save more than one-third the estimated cost. The necessary surplus capacity of a reservoir is the greater as the place to be protected is the farther down stream from it. Therefore there occurs, even in great undertakings, a much diminished effectiveness in scores of miles of intervening distance, a reduction which becomes fatal in a few hundred miles, and in correspondingly shorter distances when the volumes involved are less or the intervening affluents are the more considerable in size.

A perfect system of operating a reservoir is sometimes impracticable. Even in case of the single purpose of flood protection it is sometimes difficult in the rainy season to decide whether to empty a reservoir whose discharge may unite with a flood moving down another tributary and so cause greater damage, or to hold the contents and risk a flood finding the reservoir already full. The reservoirs above Dayton, Ohio, containing 2,600,000,000 cu. ft., were full when the great storm came which caused the destructive flood of 1913. If the storage is to serve several purposes, its control is much more complicated. Fortuitous conditions may sometimes favor all interests, but they are as likely to be adverse. The flood protection of a valley would imply that a reservoir be emptied as soon as is practicable after a flood crest has passed, in order that its volume may be available to intercept the next flood which may come at any time. Water supply for municipalities or power

purposes requires, on the contrary, a reservoir always as full as possible, whose discharge shall be a regular amount at times of less than average flow. Navigation interests demand the retention of the full reservoir until a low water stage occurs so that a maximum increase of volume and stage may result from its full utilization at this time. Thus, such uses are far from identical, and a storage reservoir is usually operated in a way to favor the predominant interest concerned, or else its capacity must be enough greater to satisfy all the different utilities involved.

19. Typical Characteristics of River Channels.—Formed by the confluence of its headwaters and augmented by its tributaries, both small and great, a river normally presents a succession of variations in depth and width, velocity and volume, direction of flow, slope of water surface, and condition of bed and banks which characterize natural waterways. Fig. 8 (p. 70), mapping a portion of the Mississippi about 40 miles below St. Paul typically illustrates the curving of the banks successively to the right and to the left, and the changing widths of the stream. Fig. 9 (p. 71), giving the profile of Fig. 8 through the deepest water at a 5-ft. stage, exhibits the varying channel depths. Fig. 10 (p. 72), shows cross-sections at A–B, C–D, E–F and G–H of Fig. 8, which indicate the usual irregularity, modification of form and variation of area of such successive sections. The area of each of the two last mentioned cross-sections, which lie within the influence of the sharply curving part, is about 20 percent greater than that of each of the two first named, which occur where the curvature is slight; inasmuch as the volume of flow is constant, there being no affluents in the distance considered, the mean velocity in the sharply curving portion is correspondingly less than that above. The length of a river, measured along its winding channel, is often two-thirds or three-fourths greater than the air-line distance between the same extreme points; and sometimes the channel length is double, as in the case of the Mississippi where the distance by river from Cairo to New Orleans is 965 miles, while it is little more than half this, direct.

We may, then, consider that a river normally consists of a succession of pools and shoals. The pools occur where the banks are curved, with the deep water near the concave bank and the depth tending to increase with the degree of curvature. The shoals exist at those places where the curvature of the stream changes in direction, thus constituting the crossing from the deep water near the concave bank on one side to deep water at the concave bank on the other

side of the river. At such a crossing there occurs a dissipation of the energy of flow (which in the concave curves has concentrated to main-

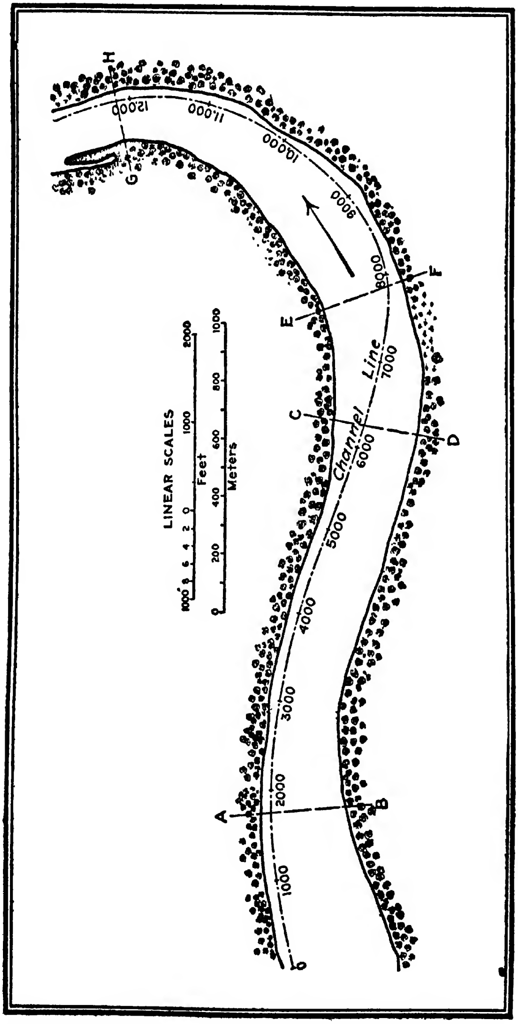


Fig. 8.—A typical portion of a river.

tain the channel) resulting in that shoaling of the river and that instability and uncertainty of channel conditions over the bar, which limit

the navigability of the river, whose improvement is therefore the direct object sought. In a general way the length and cross-section of the pools vary with the size of the stream, the larger rivers having the larger pools; and yet their dimensions on any one stream vary enormously through the equally important effects of influences other than volume, such as velocity, sediment, and material of the river bed. Successive pools are separated occasionally by only a slight shoaling of the water; usually a considerable bar intervenes; and sometimes very shoal crossings occur; and these, again, vary greatly in length, shallowness and tortuousness. Illustrating the limiting effect of the crossings it may be stated that, while there are numerous bars of only 6 or 7 ft. natural depth in the 600 miles of the Mississippi River between Cairo and Vicksburg, the average depth at low water between these two cities is 35 ft.; while it is twice this below Vicksburg.[1]

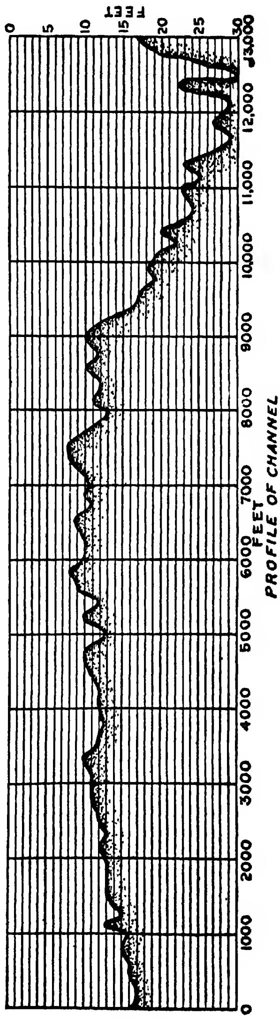


Fig. 9.—Depths along channel line.

At low water the surface slope of the pools is less than across the shoals; but at higher stages of the river this slope in the pools becomes relatively greater and that over the shoals is reduced until, at flood heights, the difference of rate of fall is largely obliterated. This

[1] Document No. 50, H. R., 61st. Congress, 1st. Session.

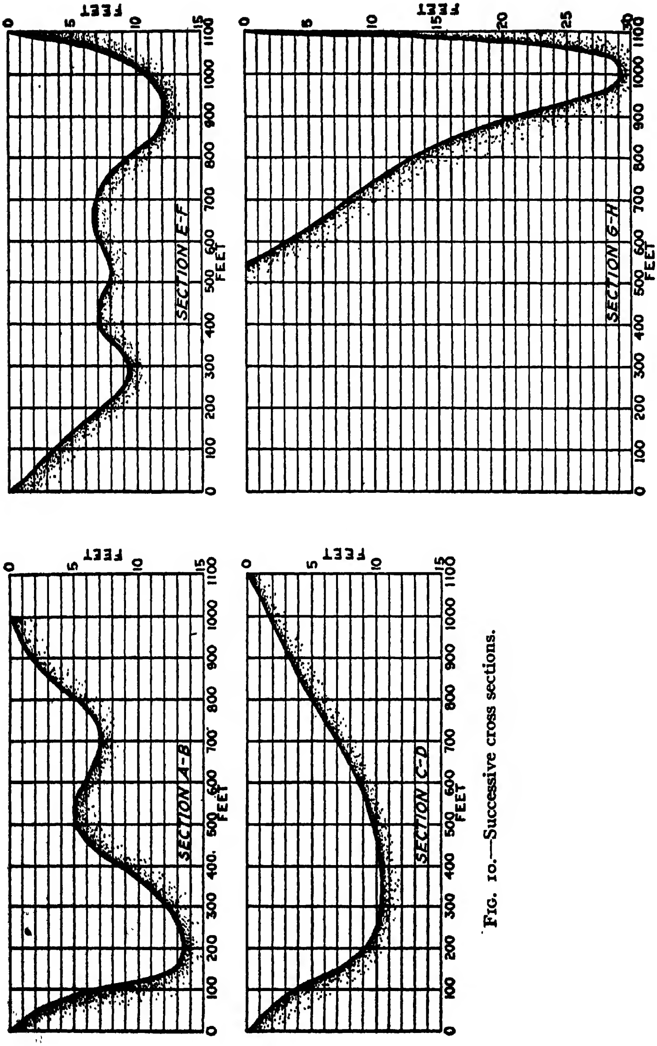


FIG. 10.—Successive cross sections.

gradual increase of surface slope of the pools, which occurs as the volume of flow is augmented, with the usually accompanying greater hydraulic radius, produces a corresponding increase in the mean velocity; the resulting tendency is to further deepen the pools as the river rises. Similarly, the influence of the reduced slope over the shoals is to decrease the average velocity there, with a resulting tendency to raise the crest of the bars. This influence is, however, usually opposed by an accompanying increase of mean depth; the tendency of which is to lessen the effect just mentioned, sometimes to neutralize it, and often to reverse it when the proportional increase of hydraulic radius caused by the higher stage of the river is greater than the relative decrease in value of surface slope at the same place. As the high waters subside the opposite effects are produced, and the river returns to approximately its original condition on again reaching the stage of water that then existed. Such a cycle of temporary variations is to be normally expected and is not particularly serious unless permanent changes result. It is therefore evident that, in streams with unstable beds, the depths at any stage are not at all equal to the depths at low water increased by the rise to that stage. This is strikingly shown by the remarkable fact that a rise of 10 ft. in the Colorado River at Yuma in 1909 was accompanied by a scour of 30 ft., or a total increase in depth of 40 ft.[1]

These typical, general effects produced by high water are accompanied by others whose bearing upon the planning of works of improvement is sometimes negligible and sometimes great. The position of the channel across the bar is likely to vary as the river rises or falls, because of the varying velocity and direction of current produced by the changing conditions. There is often danger that the swirling surges of high waters, so different from the comparatively quiet flow of the low water stream, will cause such erosion or deposit as to radically change those conditions of river bed and banks whose stability is particularly essential. At flood heights, overflows upon the adjacent lands frequently occur under circumstances which require the theoretical separation of the stream, for the intelligent application of hydraulic formulæ to the study of its control, into two parts—that section between its natural banks, and the overflowing portion; because, if the channel section were not considered separately, the influence of erratic conditions in the overflowing portion would have so irregular and uncertain an effect upon

[1] H. T. Cory, "Irrigation and River Control in the Colorado River Delta," Transactions American Society of Civil Engineers, Vol. 76, p. 1214.

the values of the terms of hydraulic equations that it would be impossible to reason from them to their results upon channel improvements. High waters also frequently cause such violent changes in direction and conditions of the currents, when the flood crests flow across points or submerged lands in their rush to escape, that most serious effects are produced upon channel conditions and upon the efficiency of regulating works; so that the confinement of the flood waters from extensive overflow by means of levees is often a necessary auxiliary to channel improvement. Tributaries entering the river, with their enormous changes in ratio of volumes, may cause conflicting current and velocity effects of such great variability as to further complicate an already intricate proposition while the effect of their influence is undergoing assimilation by the main stream.

20. Hydraulic Principles Govern Stream Flow.—Approximate as are the ordinary hydraulic computations, made for the purpose of determining the dimensions and other details of conduits, flumes and other regular channels, there is such a degree of certainty attending the consideration of artificial channels that their design is comparatively simple and definite. Natural waterways have, however, several characteristics of much greater uncertainty. The especial source of complications is the very great differences in the physical characteristics of the material composing the bed and banks of rivers, particularly displayed by its infinitely varying resistance to erosion and transportation by the current; and this variation is the fundamental cause of the changing slope, width, depth, velocity and curvature at successive sections which the stream has gradually attained under the varying resistances encountered as it flows onward, thus forming the sequence of pools and shoals whose characteristics must be mastered in complete detail, especially at the low water stage, as an essential prerequisite to the planning of improvements. Although much more complex, yet this "non-uniform flow" of natural waterways at a constant volume is equally subject to law, the principles of hydraulics forming here also the ultimate basis of the theory of stream flow. The standard hydraulic formulæ giving the relations between surface slope, mean depth, velocity, area of cross-section and volume are used in estimating the hydraulic effects of all those features of proposed works of improvement which are planned to modify the natural condition of the river. Further, when we note the added uncertainties introduced into the hydraulic studies by the endless fluctuations in the stage of the river, as the contributed waters swell or diminish the volume with the accompanying changes

in velocity and other characteristics which may produce alterations of the channel tending to prevent a return to original conditions when the low water stage is resumed, it will be realized that the intricacies of non-uniform flow are seriously increased by the complications of variable volume of discharge. The involved hydraulic investigations, employed in planning the needed low water depth and width at shoals, must not obscure the equal requirements that the channel obtained shall not be tortuous or have difficult turns, and that the works of improvement shall not impede navigation at higher stages of the river nor cause destructive erosion or overflows. Thus such hydraulic investigations have not the few and comparatively simple computations appertaining to artificial channels; but they involve many, intricate, interdependent determinations whose various essential results must both harmonize with each other and be individually satisfactory in their indicated effects. In all these computations the use of Chezy's formula, $v = c\sqrt{rs}$, and of its complement, $Q = av$, with correlated formulæ, constitute the usual basis of the complex study of the hydraulics of river regulation. Always the significance of the various computed values is to be keenly interpreted in regard to their effects upon depths, widths and capacity to produce erosion, transportation and deposition. That unexpected irregularities of results attending the inauguration of such projects may be avoided, it is usually wise to reduce the variability of velocities to its practicable minimum; as this is the best single criterion of the elimination of those great or abrupt changes which are the general cause of unfortunate results. In order that the mathematical values derived from the formulæ shall form a reasonably reliable basis for engineering judgment in predicting the effects of projected design, it is not only necessary that careful surveys shall determine the magnitude of the known factors involved, but it is especially requisite that extreme caution and care be exercised in deciding what value shall be given to the coefficient "c" of the Chezy formula in each case of its use. Kutter's value for this is sometimes the most reliable and is frequently employed; while Bazin's value is advocated by various authorities; but by far the preferable and most reliable method is to determine the value of this coefficient from field observations including slopes, cross-sections and gaugings. In any case, local conditions as observed should be given much weight in the mathematical argument involved, every step of which should be guided by good judgment.

21. Incompleteness of the Filamental Theory of Flow.—There is a phenomenon of flow whose apprehension is necessary for an adequate comprehension of the complex action of flowing water and the effects produced by it in channels of earth, sand or gravel. It is the principle variously called "vortical," "tumultuous," or "sinuous" flow. In this conception one must imagine a minute, elementary volume of the stream and follow that particular particle in its devious movements. The direction of its course is exceedingly varied, first to the right and then to the left, now toward the surface of the river and next toward the bottom, as it at the same time is progressing down stream as a component element of the aggregate flow. In this complexity of movement of the component particles, each one is responding to physical law; the motion at any instant being the resultant effect of its existing energy, of gravitation, of impact imparted by other particles of water touched in its course, and of the reaction from the bed or banks when its path encounters these limiting surfaces. As stated in ordinary hydraulic discussions the impelling force is gravity, causing a general flow in the direction of the surface slope; but such published works usually assume that the stream is an aggregation of imaginary threads of water moving parallel to its axis, thus forming the conception of filamental flow of the books, whose especial advantage consists in offering a consistent interpretation of the river's progressive movement only; and it thus ignores the infinitely varying movements of the component particles whose aggregate flow-effect is, nevertheless, expressable in terms of the usual hydraulic formulæ.

A typical particle of the stream is thus actually moving, at any instant, in a direction usually inclined to the axis and so not normal to a right section of the river; and its actual instantaneous velocity and direction may be analytically represented by its components in the sectional plane (right or left, and up or down), and in the direction of the axis (down-stream, or possibly up-stream). At successive points in its course its direction and velocity will vary, and its component values will be correspondingly different. If we imagine these three component values to be determined, instant by instant, for all the particles successively passing any single point in the cross-section of the stream during a considerable length of time, and the average value of each of the three components be derived, then it will be found that the right and left components usually will approximately balance; the vertical components will also approach a zero summation; and the axial component thus averaged will have that

velocity value ascribed to the imaginary filament, of the ordinary theory, which coincides in position with the point assumed. Therefore the usual conception of parallel filaments in a stream does give a reasonable mathematical theory of the onward flow itself, but omits consideration of the more fundamental, actual movements of the countless individual particles whose limitless variations in direction and velocity produce other effects of great moment. One of the earliest of the scientifically discriminating discussions of this definitely existing internal turmoil of water flowing in rivers is reported in the Proceedings of the Royal Society of London, Vol. 28, pages 120–125.

Optical proof of the ultimate vortical character of stream flow is not usually practicable in this transparent fluid. However, a discriminating observer will see corroboration of this irregularity in onward flow in the occasional swirls and eddies, "sucks" and "boils," riffles and rips, which may be seen when the vortical action is pronounced or occurs in aggregated masses; or it may be detected in less violent cases when a small submerged object, having about the same specific gravity as the water carrying it, marks an irregular course in response to the summated impact effects of the particles of water striking it.

Although the vortical theory satisfactorily explains the absence of those destructive velocities[1] and consequent enormous difficulties of utilizing streams which would exist if real filamental flow were an actual fact, the definite purpose of its discussion here is to designate it as the particular agency of erosion. When we apprehend the particles of water at the bed and banks of a river, not as flowing filamentally parallel and so acting through frictional effects only, but rather massing and striking the earth in that irregularity of movement which actually exists and so producing real impact effects combined with friction, we can better comprehend the resulting disintegration of the bed and banks which takes place in all rivers. Statements of authorities, such as the following, which are quoted from the paper of E. H. Hooker entitled "The Suspension of Solids in Flowing Water" in Transactions of the American Society of Civil Engineers, Vol. 36, pp. 239–340, indicate the conviction of various scientific investigators that natural streams necessarily have this vortical character of flow: "Flowing water is known to be actuated by a vast number of internal currents or vortices." "Observation readily detects whirls, boils and eddies in the act of bringing water and

[1] See Encyc. Britt. (1910) Vol. 14, p. 35.

suspended material from the bottom to the surface and laterally from the sides to the center of the river." "This upward inclined eddy motion does not in any way conflict with the general horizontal flow of the stream. The whole motion can be perfectly understood by comparing it with the identical phenomena observed at an ordinary campfire in the open plain; the smoke, wafted by a gentle breeze, rising upward in the form of an inclined eddy, expanding as it rises." "Partiot emphasizes the idea that the sands are only sustained by eddies and vortices."

Another important phenomenon of stream flow, which directly affects the channel developing process of the currents in sedimentary soil, is that known as transverse or cross-currents. This term, of course, does not refer to their actual direction of flow but rather to their lateral and vertical components, their longitudinal components constituting the ordinarily considered filamental currents flowing parrallel to the hydraulic axis of the river. The magnitude of these transverse components is generally quite small compared to the axial; but as they are very definite and continuous in their action, their general effect is very important. They necessarily result from the conditions of curvilinear flow existing in the bends of rivers, where the inertia of the flowing water causes radial dynamic pressures. Professor James Thomson in 1876 pointed out[1] that because of the centrifugal force of the stream flowing in a curve with the resulting increase of elevation of water surface at the concave bank above that at the opposite side, together with the fact that the lower part of the river section has smaller velocities than the upper portion, there must occur a general movement of the water from the concave margin toward the convex one at the bottom of the river, with an accompanying downward movement of the water in the concave bank and an upward motion near the convex side of the stream, and a simultaneous transverse flow from the convex toward the concave margin in the upper portion of the section. The next year he offered experimental proof of this secondary current flow. Ten years later Professor Reynolds indicated[2] the significant effect upon erosion and sedimentation which these cross-currents must have, carrying the entrained silt and sediment from the bottom of the river toward the convex bank at the same time that this material is being swept much more swiftly down-stream. More recently these characteristic

[1] Proceedings, Royal Society of London, Vol. 25, pp. 5–8.
[2] Report, British Association for the Advancement of Science, 1887, p. 557.

transverse currents have been instrumentally observed on the Garonne, the Dnieper and other rivers.

The laws expressing the character of this irregularly spiral flow, characteristic of the curved portions of rivers, are not known. The development of such laws is being studied[1] and their determination will make more definite the principles controlling the regulation of rivers. The particular reason for this expectation is that the nature of this transverse movement in curved portions is such that it performs a significant part in the movement of sediment to its prevalent place of deposit at the convex side, and affects materially the erosive process at the concave bank.

22. The Significance of Erosion, Transportation and Accretion.— The ultimate difficulty besetting projects of river improvement is the instability of the stream caused by the fact that its bed and banks are composed of a material that is erodible; for while the instability is partly a matter of direct erosion and partly of deposition, even in the latter case the alluvium is largely derived from the eroding banks. Whether or not the difficulties of the situation in the navigable portion of a river are materially affected by the water-borne detritus of its headwaters and branches is an open question. That this effect is generally produced is strongly contended in Professional Paper 72 of the Geological Survey (1911), in which the general process is thus stated:

"The removal of the forest on steep slopes generally increases the tendency to erosion. This increase may be very slight if the land is kept well sodded or if the soil is of a certain porous or stony type, but in a region like the southern Appalachians, with its deep soil and abundant, often torrential, rainfall, erosion is generally more rapid—it may be very much more rapid—on cleared than on forested slopes. Erosion once begun, as a rule, soon develops gullies that furnish so much sand, clay, and cobble to the streams that they become overloaded and are unable to carry away all the waste that is brought to them. The excess waste is therefore deposited first in the channel, until that is practically filled, and then over the alluvial flood plain, which is thus converted into a barren waste of sand or loose stones. The waste then begins working down-stream, filling dams and pools as it goes, and soon gets down into the navigable parts of the great river systems, such as the Tennessee, making more difficult the problem of maintaining navigable channels."

The preceding quotation (from a discussion which particularly treats of the relation of the forests to the preservation of the soil

[1] *e.g.*, Annales des Ponts et Chaussées, 1913, I, pp. 112–133.

whose conservation is so vitally important to agricultural prosperity that the prevention of its loss is fundamentally justified by that predominant interest alone) places particular emphasis on deposit resulting from the overloading of the stream by detritus. Now it is true that for any unvarying condition of a river with regard to its cross-section, slope and velocity there is a certain maximum burden of detritus which it can carry, and if this is exceeded from any source the excess burden will be deposited; as discussed in paper 2826 of the Institution of Civil Engineers (Vol. 119) for a silt-carrying stream with bed and sides of similar material, flowing in a condition of silt equilibrium, R. G. Kennedy found for the special conditions existing that the critical mean velocity at which neither erosion nor deposit occurs is practically equal to $0.84 \sqrt[3]{(\text{depth})^2}$, and for heavier silt the coefficient becomes gradually larger, and the exponent may vary somewhat, but probably little. The fact is that as far as investigations have gone, rivers are not always found to be carrying their full load capacity of sediment.[1] The fundamental conception of this phenomenon is that of most hydraulicians, as expressed in Encyc. Britt. (1910), Vol. 14, pp. 77–8:

"If in one part of its course the velocity of a stream is great enough to scour the bed and the water becomes loaded with silt, and in a subsequent part of the river's course the velocity is diminished, then part of the transported material must be deposited. Probably deposit and scour go on simultaneously over the whole river bed, but in some parts the rate of scour is in excess of the rate of deposit, and in other parts the rate of deposit is in excess of the rate of scour. . . . . If a river had a constant discharge it would gradually modify its bed till a permanent régime was established. But as the volume discharged is constantly changing, and therefore the velocity, silt is deposited when the velocity decreases, and scour goes on when the velocity increases in the same place. When the scouring and the silting are considerable, a perfect balance between the two is rarely established, and hence continual variations occur in the form of the river and the direction of its currents. In other cases, where the action is less violent, a tolerable balance may be established, and the deepening of the bed by scour at one time is compensated by the silting at another. In that case the general régime is permanent, though alteration is constantly going on. This is more likely to happen if by artificial means the erosion of the banks is prevented."

Although this view omits consideration of detritus contributed by the affluents of the river, it is characterized as the truer funda-

[1] *e.g.*, Report, Chief of Engineers, U. S. A., 1875, pp. 966–7.

mental conception because it directs particular attention to the predominant influences of the case, while the effect of the contributed detritus is primarily local and becomes gradually general only through the operation of the principles summarized in the last quotation. The continually active and controlling consideration is the taking up, carrying forward for a distance, and the dropping of sediment by the stream because of the varying velocities at successive portions of the river caused by variation in section and slope, and by fluctuations of the stage of water everywhere. This movement is thus expressed in the case of the Mississippi River, in "Report, Chief of Engineers, U. S. A.," 1896, p. 3448:

"An immense amount of earthy material is constantly being moved by its flow, the most of which material is derived from the tearing down of its banks. The material torn from its banks is moved along in its channels, forming obstructions to the flow of the flood waters and lessening the navigable depths at many places."

Of secondary importance is the question of contributed detritus whose effect locally may be considerable, but which rapidly becomes reduced; yet it can hardly be considered negligible because its influence must persist to some extent in the intermittent progress of the sediment down-stream. This effect is always detrimental because the increase in the load of silt tends to adversely affect the régime of the river directly by adding to its shoal places and indirectly by the increase of deposition affecting slopes and cross-sections and so the velocities. To what extent this harmful effect occurs is a question varying with each stream and each section of a river, but which is believed to be usually of minor importance.

While the mineral matter carried by rivers in solution is often considerable, sometimes amounting to about one-third as much as the weight of the mineral matter in suspension in very turbid streams, this has no direct significance in the question of the improvement of rivers for navigation. It is the burden of suspended matter which has a particular import, as it constantly varies with the locality and stage of river in consequence of conditions favoring erosion at one place and deposition at another, and locally with the contributed silt of affluents. Some instances of heavily burdened rivers are the Colorado, carrying an average by weight of 1 part in 142 of silt; the Missouri, 1 : 265; the Rio Grande, 1 : 291; the Po, 1 : 900; the Mississippi, 1 : 1500; the Rhone, 1 : 1775; the Nile, 1 : 1900; and the Danube, 1 : 2880. The actual quantity of detritus thus moved is enormous; the Colorado River has been found to transport past Yuma more

than 100,000,000 cu. yd. of sediment per year, its mean annual discharge being about 1700 cu. ft. per second; and the Mississippi River, with a mean discharge of about 620,000 cu. ft. per second, annually carries into the Gulf more than 400,000,000 cu. yd. of such material. It is estimated that the Missouri River contributes to the Mississippi from 200,000,000 to 400,000,000 cu. yd. per annum, while the yardage from caving banks of this river in the state of Missouri alone is about twice that amount; similar erosion in the Mississippi between the mouths of the Missouri and Ohio Rivers yielded over 60,000,000 cu. yd. per annum thirty years ago, but only about 80 percent as much since a considerable revetting of the banks has been accomplished; and nearly 900,000,000 cu. yd. is annually contributed by the caving Mississippi River banks between Cairo and Donaldsonville.

As the velocities in a river section are typically a maximum toward the concave side from the center of the channel, these banks are especially subject to a progressive disintegration that may be rapid, slow or negligible, depending upon their resisting capacity and upon the velocities of the attacking currents. While the actual conditions attending erosive action often do not in their details agree with formulated theory, probably because their consideration has been generally based on the filamental conception of flow without reference to the variable vortical velocities which are the direct agency, yet the observed facts given in Table No. 2 will give a fair average idea of the relation existing between the velocity of the stream and its capacity for producing erosion. The values are those of the bottom velocities at which movement begins for different materials, as determined by Bouniceau in 1845.

TABLE NO. 2

Clay	0.26 to 0.49 ft. per second.
Coarse sand	0.72 to 0.98 ft. per second.
Coarse gravel	0.36 to 2.00 ft. per second.
Ordinary pebbles	2.19 to 3.28 ft. per second.
Stones (size of an egg)	3.28 to 3.94 ft. per second.
Conglomerates	4.99 ft. per second.
Sedimentary rock	6.00 ft. per second.
Solid rock	9.84 ft. per second.

Dubuat found that the velocities at which transportation in complete suspension begins were generally from 25 to 50

percent greater than those given in the above table for the values producing the commencement of a sliding movement.

It is thus seen that the heavier the particles of the material may be, the greater is the velocity of current necessary to move them from their position. As the greater velocities occur during high water, the erosion of the concave banks of the pools is particularly rapid during floods. This disintegration of river banks not only occurs through the direct action of the current, but is accelerated by the loosening and falling into the stream of overhanging masses of

FIG. 11.—Protecting an eroding bank.

earth which the current has directly undermined; and it is especially severe on alluvial streams when the river is falling from a flood stage which has saturated the banks, resulting in a reduced cohesiveness of the earth and an increase in weight which combine to produce extensive sloughing when the supporting hydrostatic pressure of the water is gone. So serious is erosion that its prevention is generally an essential part of works of improvement, required to secure the stability of conditions necessary for definite and permanent results. A view of a caving bank is shown in Fig. 11.[1]

Of the material eroded, usually the greater part, consisting of the smaller, lighter particles, is carried in suspension by the stream as long as a sufficient velocity is maintained, and the larger, heavier fragments are rolled and dragged along the bottom until they encounter a velocity so reduced that it is no longer sufficient to move

[1] From Journal of the Association of Engineering Societies, Vol. 40, p. 120.

the material, or is so increased as to carry it into actual suspension, while material of intermediate size is often rolling along the bed at one moment and at another is carried into temporary suspension. The dragging motion is so pronounced during the high waters that sometimes the drift of gravel in even navigable streams is distinctly audible. There is also evidence that the bed of alluvial rivers maintains a very slow but persistent progress down-stream, this movement of course being a maximum at the surface of the bed, but actually penetrating downward to a depth of several feet. A peculiar occurrence attending the movement of sediment at the bottom of an alluvial stream is that of sand waves, consisting of the formation of alternate ridges and hollows, transverse to the direction of flow, which increase in size and rate of movement downstream as the bottom velocity increases up to a critical velocity depending upon the size and weight of the moving particles of sediment; but for greater velocities the size and progressive travel of these sand waves decrease. A typical description[1] of a study of such a phenomenon made on the lower Mississippi River states that:

"At high water, in the shallow sections, the average height was about 13 ft., the distance between crests about 450 ft., the rate of travel about 22 ft. a day. In the deep sections, the height of waves at high water was about 7 ft.; length, 330 ft.; travel, 40 ft. a day. At lower stages the waves are not usually so high, the distances apart are shorter, and the rate of travel slower; but the movements are still of considerable magnitude. Even in very shallow water there were found numberless small sand waves, 1 ft. or 2 ft. high and less than 100 ft. apart."

Such a condition is interesting, not only as illustrating the manner in which the rolling fragments progress along the river's bottom but also as proof that variation may occur in the perimeter of a river section which is ordinarily assumed to remain fixed, and so sometimes introduces serious error in gauging computations.

The condition of a river, then, with regard to the silt and sediment carried by it, is one of unstable equilibrium; the inequality of velocities in different portions of each cross-section and particularly their variation from section to section causing such a succession of changes that at one moment there is a tendency to carry still more or heavier material because of increased velocities encountered, and soon after to deposit some of its suspended matter when the burden

[1] William Starling, on "The Discharge of the Mississippi River," Transactions, Am. Soc. C. E., Vol. 34, p. 426.

is found to be too great for its reduced sediment-bearing capacity. When we recollect that the power of a stream varies (in the capacity to transport sediment) approximately as the $\frac{5}{2}$ power of its velocity,[1] or the weights of fragments transported vary as the sixth power of the corresponding velocities, it becomes very evident that small changes in velocity cause relatively great changes in the character and amount of the sediment carried.

The mathematical law just expressed has an equal import in the consideration of accretion, which is the third phase of the sequence, the deposit of sediment occurring when the energy of the current, as indicated in the preceding paragraphs, becomes insufficient to move further the eroded material. A rather striking consequence of the principles involved in the formation of such deposits is the observed fact that, at any certain falling stage of water, rivers are depositing sediment of one size at one place and of another size at another, as a gravel bar building up where the lessened velocity is still considerable and a sand bar forming where the velocity is correspondingly less; and the additional fact that, as the velocities at each of the bars continue to reduce, the size of grain of the deposited sediment becomes proportionately smaller as the shoal is built upward. The values given above, to show the general relation between velocities and transporting power, are of course also applicable here as indicating the limiting values of depositing velocities.

From the nature of the case, the sediment of any accretionary deposit comes from varying distances up-stream from the location of its formation; some from the eroding concave bank opposite and immediately above, and a portion from each of the eroding sections of the river for an indefinite distance up-stream, in entirely irregular and unknown proportions as the material is loosened, transported and deposited particle by particle in a new position; while a portion, very small ordinarily but sometimes considerable immediately below tributaries, may come from the soil-wash of the fields and the erosion of the branches themselves.

A typical illustration of the general relative effects of the progressive erosion at one bank and accretion at the opposite one is shown in Fig. 12 (p. 86), the dotted lines of which show the banks in 1882–83 and the full lines the positions of the same river banks in 1895–96 of Springfield Bend and the bend below of the Mississippi River about 10 miles above Baton Rouge; but, as just indicated, the sediment is not transported directly across the stream from one bank to the other,

[1] Proceedings, Inst. of Civil Engineers, Vol. 119, p. 285.

but is derived from indefinite distances above the accretion, in entirely untraceable identity of origin. It is this fact which forms the

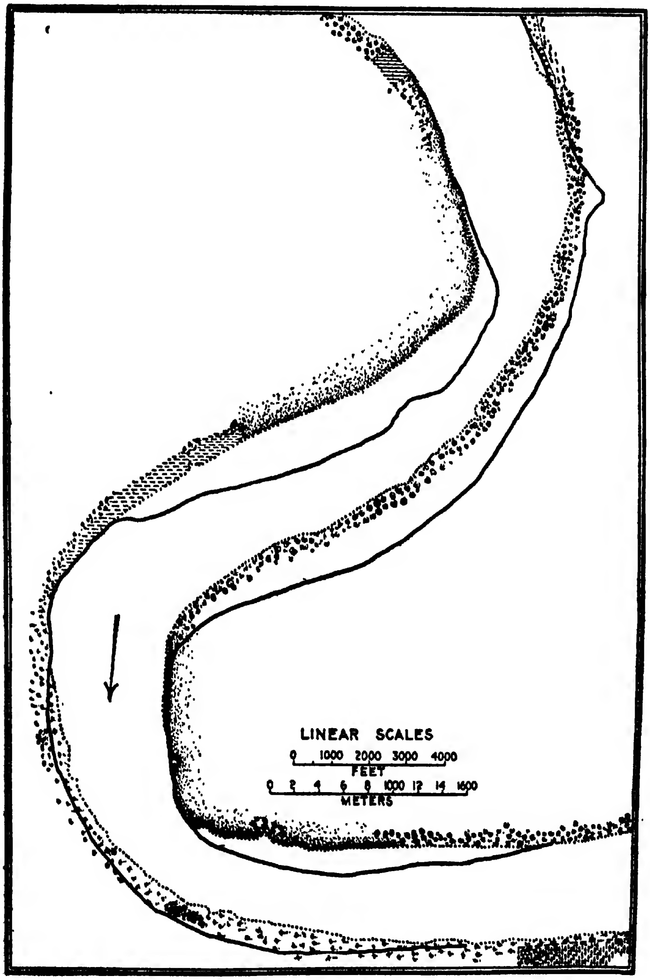


FIG. 12.—Effects of erosion and accretion.

basis of the legal rule that accretions become the property of the owner of the expanding bank, while land which finds itself on the

opposite side of the river through its washing for itself a new channel across a narrow neck of land, thus forming a "cut-off," still belongs to the original owner because its identity is preserved. Notwithstanding the fact that the immediate effect of such cut-offs is to shorten the length of rivers, they should be generally prevented because the great increase of surface slope, caused by encountering the same fall in a reduced distance, produces so violent changes in velocities and consequent instability of conditions as to constitute a serious situation which gradually extends for considerable distances up-stream and down, but which lessens in the severity of its disturbances as it extends.

Erosion, and the accompanying accretion, take place intermittently, and yet so systematically and persistently that the modifications thus naturally produced in a term of years may be foreseen with considerable definiteness. Compared with rivers in piedmont regions, alluvial rivers have a relatively small surface slope, but their hydraulic depth is generally greater; so that their velocities, especially at high water stages, are usually considerable. The alluvium of their banks and bed is generally so easily erodible that the progressive development in the change of the course of the stream is rather rapid. This is indicated in the preceding figure, and still more prominently in Fig. 13 (p. 88), showing a section of the Mississippi River about 20 miles below Vicksburg. Fifty years ago the channel occupied "Palmyra Lake"; but in 1867 a cut-off was formed across the narrow neck intervening and this great loop was thus transformed from the main stream into a lake which is so characteristic of alluvial valleys. Since that time the river has rapidly been changing its position by eroding its concave banks and adding to the convex banks. The course of the river in 1882–83 is shown by the dotted lines, and its position thirteen years afterward by the superposed continuous bank lines. The gradual progression in erosion and accretion from the comparatively gentle curvature of the earlier date to the sharper curvature of the later time is plainly indicated.

Piedmont rivers have, however, so different a soil, consisting at such frequent intervals of gravel, ledges and sometimes solid rock, and with banks generally of clay or a more resistant material, that the progressive changes are comparatively very slow and irregular.

From the general discussion of this chapter it is readily perceived that a fundamental necessity of river improvement is to secure as thorough and definite a stability of conditions as it is practicable to attain.

In connection with the detailed discussion of various types of works of river regulation in succeeding chapters, there are given many illustrations of natural conditions which coincide with the

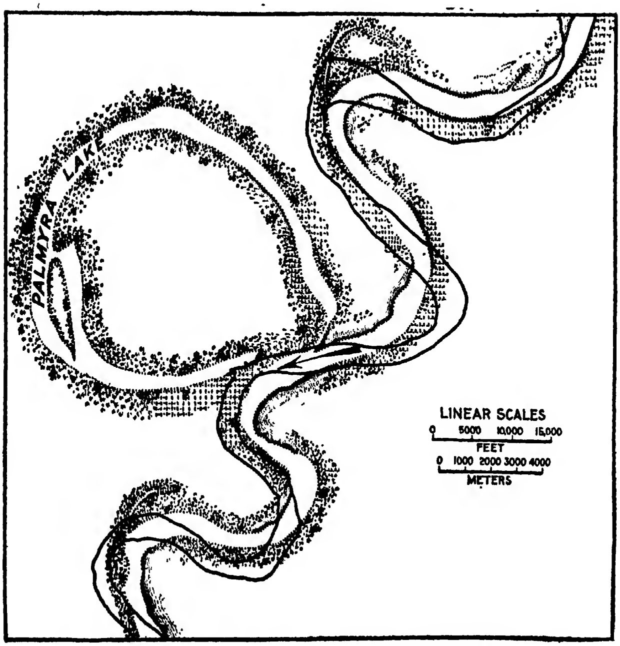


FIG. 13.—Changing channel of an alluvial river.

typical principles discussed in this chapter; as well as instances of variation from the ordinary type where unusual circumstances produce uncommon effects.

CHAPTER III

INVESTIGATIONS, SURVEYS, ETC.

23. Methods Employed in Making Topographic Surveys.—Besides the commercial considerations indicated in the first chapter and some of the general conditions indirectly affecting stream flow as discussed in various parts of the second chapter, which require a thoroughness of investigation proportional to the significance of their bearing upon each project, there are many particular facts, which definitely characterize each river, that must be directly determined by surveys.

Because floods generally swell the volume of a stream until it overflows the valley lands, it is desirable that a topographic survey be made of both sides of the river from the immediate banks of the stream to the high lands which mark the extreme limit of overflows. These are needed for such collateral purposes as the determination of high water effects upon the channel conditions, the intelligent location of levees, or the extent of probable damage resulting from certain works of improvement which tend to raise the water surface at times of flood; and by making topographic surveys at successive periods, the changes wrought by erosive action and accretion, at the river banks, are definitely determined.

The planning of such a survey of a river valley depends upon various factors, such as the magnitude of the stream and valley; the degree of importance attaching to the high water effects; the nature of the commerce to be provided for; the probable character of the improvements; and the degree of accuracy that the results should have. The survey of a great river, like the Mississippi or Ohio, involving various and extensive improvements and accommodating a general commerce, must be much more extensive and precise than that of a relatively small and unimportant stream, like the Clinch, Oconee, Minnesota, or Willamette River, whose works of improvement are comparatively simple in character and made for a commerce of moderate amount, largely consisting of logs and timber rafts.

A thorough and complete topographic survey of a river valley is controlled by a geodetic survey consisting of a triangulation system extending from end to end of the valley in question, with its

stations generally on the bluffs at each side of the valley at advantageous points above the highest floods; and one or two lines of precise levels also extending through the valley, with bench marks of accurately determined elevation established every mile or two, preferably at or near triangulation stations. The permanency of such stations and bench marks is necessary to insure the definite determination, by successive surveys from time to time, of the comparative conditions, tendencies and actual changes occurring in the river.

Based on this geodetic framework, the topographic survey is made by either the transit and stadia method or the plane table method, in approved details as given in standard text-books on surveying; but, unless the valley is largely open ground, the stadia must be the chief reliance either by directly using that method or by largely employing the telemeter with the plane table alidade, because of the greater facility of thus making locations through obstructing trees, bushes or cane. The azimuth of each topographic line is taken from and is checked at its end by that of the triangulation system, and its accuracy is checked by computing the error of closure; while the accuracy of the topographic elevations is controlled and checked by closure upon the level bench marks.

It is generally wise to have the topographic parties locate appropriate instrument stations for the accompanying hydrographic survey on the immediate banks of the river as their work advances; or, if the hydrographic parties precede the topographic, the latter must carefully locate the instrument stations that the former have occupied. Care should also be taken to obtain the date, position and especially the elevation of all available high water marks that are authenticated by record, by reliable witnesses, or by direct observation of the evidences remaining for a time after the passing of the flood crest; such indications as driftwood or grass caught in the forking branches of trees, or the extreme limit of discoloration on tree trunks by the sediment of the flood waters are pertinent evidences of the desired information.

To illustrate concretely some of the controlling facts of such a survey reference may be made to the following data of the survey of the Illinois River with reference to the proposed 14-ft. waterway in 1902–04 as stated by F. C. Woermann.[1] The secondary triangulation extended from Chicago to the mouth of the Illinois

[1] From thesis submitted to Washington University in 1906 for the degree of Civil Engineer.

River, involving 144 stations usually located on the top of the river bluffs and spaced from 2 to 8 miles apart; the allowable error of closure was fixed at six seconds, and about half the angles actually closed within three seconds or less. The tertiary triangulation required 1115 stations usually established on the immediate river bank at an average distance apart of about 2000 ft.; the established limit of closure of angles as measured by the repetition method was twenty seconds, and the actual error was usually between two and ten seconds. The length of duplicated precise level lines was 343 miles, on which 139 permanent and 488 temporary bench marks were established; the error of closure was limited to 3 mm. $\sqrt{2}$ K, and the probable error of the whole line actually was 13.55 mm. The topography covered 683 square miles of territory, necessitating the occupancy of 17,719 stations; the limiting error allowed in distance was 1 in 500, in azimuth five minutes, and in leveling as carried by vertical angle computations $\frac{1}{2}$ ft. The wye-level lines aggregated 1520 miles, running on each bank of the river and connecting with all triangulation stations and precise level bench marks, with hydrographic hubs where practicable, and with topographic hubs when convenient; the limiting error of closure was fixed at $\frac{2}{10}$ ft. in 5 miles and bench marks were established on each bank at intervals of about 1 mile. In the hydrographic survey 5961 stations were occupied; and to explore the character of the river bed borings were made at about half-mile intervals, aggregating 569 in number and averaging 33 ft. below low water surface. The total cost of the secondary triangulation was $18,062; of the tertiary triangulation, $8136; of the precise leveling, $12,544; of the wye-level lines, $7965; of the topography, $47,614; of the hydrography, $15,136; of the test borings, $7797; of the discharge measurements, $3435; mapping, photo-lithographing and publishing maps, estimates of cost, etc., $55,587; or a total of $172,841. The unit costs were as follows: secondary triangulation, $55.24 per mile of river; tertiary triangulation, $36.28 per mile of river; precise leveling $36.52 per mile of duplicate levels; wye-leveling, $5.24 per mile of single line; topography, $67.01 per square mile; hydrography, $27.52 per mile of channel or $2.75 per cross-section sounded; borings, $0.62 per lineal foot of depth; field maps, $24.02 per square mile, including contouring, plotted to a scale of 1 in. to 400 ft.; final maps, $17.06 per square mile, drawn to the same scale; and publication maps, $3.81 per square mile as drawn to a scale of 1½ in. to 1 mile.

Another type of topographic survey resembles that just described in all essential particulars except that the geodetic control is omitted. This is allowable when conditions are such that it is unnecessary to perpetuate indefinitely a complete system of reference points to serve future surveys, because of the probability that such re-surveys will be needed only in small, detached portions of the river, or else only at exceedingly long intervals of time; and also where the precision of the survey can be reduced without harm to the project. Of course there results a very considerable saving in cost from the omission of the geodetic framework. A survey of this sort is suited to the valleys of rivers of moderate size, in which conditions are comparatively stable, and where the improvements will probably be of such character that there will exist no continuity of construction or such intimate relation between separated works as to require a maximum precision.

However, such a topographic survey should be so planned that errors may be minimized, and any of importance be detected and corrected in the field before the parties have advanced too far from the point where the mistake was made. For these reasons the duplication of readings, on instrument station locations, should never be omitted; skilled topographers and superior instruments are even more essential than where geodetic control exists; and approximations in topographic methods, which are often employed where their use does not produce material errors, should not be allowed, particularly those whose effects may be cumulative. Especially useful for the timely verification of results or for the detection of error are the observation upon Polaris for azimuth every second or third evening; the interchange of readings for azimuth, distance and vertical angle between instrument stations of the two topographic parties, once or twice a day; and the computation every evening of the closure of such circuits, surveyed during the day, both by latitudes and departures and by computation of elevations. Permanent reference points for future needs are established at important points, particularly at the head and foot of shoal places; these are carefully located by instrumental observation, and may be made very enduring by wedging or otherwise firmly fastening a bolt in a 4-in. hole drilled into a rock ledge, or by embedding a bolt in the top of a concrete monument where natural rock does not occur. A level-line, usually surveyed by the use of a wye-level and carefully verified by duplication of courses, is run along the river; not only for the immediate needs of the survey in furnishing a check upon the topo-

graphic circuits, and for securing exact elevations along the water surface as required in the hydrographic work, but also to establish bench marks for future use, particularly the "permanent reference points for future needs" as already mentioned in this paragraph.

On the survey of the upper Tennessee river, from its head to Chattanooga (188 miles) in 1891, the field work of topography and hydrography took nineteen weeks, during which about 170,000 soundings were taken, all located by the stadia method. For the topography, two transit parties were organized, one on each bank of the river, which checked each other by mutual closure every 2 miles, approximately; this average error of closure for these short circuits was 1 in 330. Polaris observations for azimuth were taken every 15 or 20 miles, borings made where necessary, and discharge measurements were taken at four locations. Two wye-level lines were run, one on each bank as far as possible, to check the stadia line elevations, the water-surface elevations, and to establish frequent bench marks; the computed error in closing for the total distance was 0.0035 ft. per mile. The mapping required 109 sheets, usually on a scale of 1/2000, and of course included all essential details of both topography and hydrography. The total cost was $15,000, of which nearly three-fourths was expended on the field work, or about $60 per mile of channel of river for the field operations, and about $20 per mile for the mapping, etc. Several Ohio River surveys have cost about $150 per mile of river, each.

In surveys of the smaller streams with narrow valleys, one topographic party may be able to progress as fast as the hydrographic party. In this case the procedure just outlined need only be modified with respect to the closures. In order to furnish the needed checks to the topographic work the hydrographer should, in addition to his usual duties, convert his instrument stations into points on a second traverse line by reading successively from one to another for azimuth, distance and elevation; and thus closure can be effected between this line and the single topographic traverse for the purpose of furnishing the necessary check upon the latter. In order, however, that the wye-level line may be satisfactorily dispensed with, the hydrographer should always take his readings for elevation, on the check-traverse line thus constituted, with level telescope, which is generally practicable as his hydrographic stations are preferably only a few feet above the water surface; and his transit should have a sensitive and reliable telescope bubble always in good adjustment for leveling, so that his determination of elevations may be

similar in procedure and approximate in accuracy to that which would be made by a regular level party, when long sights are corrected for curvature and refraction.

Of course, modifications of the typical methods of making the topographic surveys, which are necessary accessories of the hydrographic survey of the river, may be made as seems required by especial conditions of the valley or by particular needs of the survey. An example which is becoming more and more frequent in this country is that of the assistance of previous surveys which may be taken advantage of, to some extent, in the prosecution of the survey in question. Such modified procedure results also in a corresponding reduction in cost and in an increased rate of progress of the survey.

24. Hydrographic Surveys.—The more immediate and direct information, needed in the interest of improvements for navigation, is obtained by the hydrographic survey whose principal purpose is the determination of the varying depth of water throughout the whole extent of the investigation. As the position of the observed depths must be determined at the same time, this sounding of the stream incidentally fixes its bank lines, widths and meanderings; while the condition in detail of all shoal places and the position and extent of reefs, rocks and other dangerous obstructions must be secured with especial thoroughness. The hydrographic survey is made, then, with the object of disclosing the under-water condition of the river as thoroughly and completely as the topographic survey can determine the surface conditions of the visible valley.

Whatever may be the size of the stream that is surveyed it is customary to employ a rod for making the soundings whenever the depth does not exceed about 12 ft., and a lead-line for greater depths. In currents of considerable velocity the rod cannot be used for as great a depth as that given, and where the velocity is small it may be employed in greater depths. The rod should be a light but strong wooden staff with alternate feet marked in different colors, thus resembling an ordinary land surveyor's range-rod except that it should be 12 or 15 ft. long and its lower end should be bluntly tipped to prevent both wear on rock and penetration into soft bottom; each foot mark should be subdivided into tenths.

The lead-line being for the purpose of determining all depths too great to be reached by the sounding rod, its total length must somewhat exceed the greatest depth to be found. The weight itself is usually of lead which should be cast into a rather long and slender form to facilitate its rapid descent through the water and to minimize

its pull when being drawn up by the leadsman; desirable dimensions are 2 to 2½ in. in diameter at the lower end, perhaps four-fifths this diameter at the upper end, and a length of 8 to 12 in. For an ordinary velocity of current in depths of water of perhaps 30 ft., the lead should weigh about 12 lb.; while for a less depth or velocity an advantageous weight is proportionately less, and for greater values than those indicated its weight should be correspondingly increased. The lead-line is looped through an eye in the upper end of the lead, by the aid of leather or some other device to prevent rapid wear, and usually consists of ordinary sash cord. It is marked in a distinctive fashion, every foot of its length counting from the lower end of the lead as an origin, by tags of leather or cloth and windings of twine, in such a way that the markings will not slip nor the indicated depth be other than easy to be read by the leadsman as he observes the relation of the water surface to the lowest visible mark when the lead-line is vertical, thus reading directly to the nearest foot and estimating the tenths. For example, all the 5-ft. marks may be of a narrow strip of russet leather bound to the line by windings of stout sailor's twine which is also sewed through the lead-line; those of the 10-ft. marks may all consist of black leather similarly fastened, and with its free edge finished to one point at the 10-ft. mark, two points to indicate the 20-ft. mark, three points for the 30, etc., as used for the brass tags of the old-style surveyor's chain; while the intermediate foot marks may be simple windings of the twine. Because the lengths indicated by such a line vary with its degree of saturation and the lengthening effect of the pull of the lead, it should be soaked and stretched before marking it, and be kept in water when not in use; and, particularly, its indicated lengths must be compared with the corresponding true distances of a steel tape, before and after each half-day of sounding with it, and the results recorded for correcting the depths as originally recorded.

In the rare case of high velocities and considerable depths of water it is necessary to radically modify the method of sounding, using a wire instead of a lead-line and a weight which may reach 100 lb. or more; employing a mechanical device for lowering and raising the weight; and adapting the various details, such as the method of reading depths and of applying necessary corrections, to the character of the apparatus adopted.

Soundings are made from a boat moving with as uniform a speed and in as straight lines as is practicable, unless the depths and velocities of the stream are unusually great. A stable and unobstructed

position must be given the leadsman so that he may throw the lead such a distance to the front and also up-stream that, under the resultant of the boat's speed and the velocity of the current, by the time the lead sinks through the water to the bottom it shall be vertically below the leadsman's hand; and thus, as he draws up the slack of the line, he is able to read the indicated depth vertically, as it should be taken. While sometimes diagonal lines are run in order to save time, they result in aggregate loss because of very unequal spacing. The better plan is to have such lines of soundings cross the stream always in a direction normal to its axis; thus they will be nearly parallel, their divergence being a function of the curvature of the stream at any point. Their regular distance apart should depend mainly upon whether the lines cross shoals or pools, and upon the size of the stream; but it is also affected somewhat by other conditions. A typical spacing in the pools of a river of medium size is 400 ft., which should be generally reduced to 100 ft. at shoals. On great rivers the spacing may be double that just given, and on small rivers, perhaps half. It also is wise to run lines of soundings parallel to the current throughout the extent of the shallow channel of the river, at the same distance apart as that of the transverse lines already mentioned; this will both give a desirable check upon the depths obtained on the cross-lines and will disclose any intermediate irregularities that are liable to occur between the transverse lines on the bars, the ascertaining of whose exact condition is the most essential detail of the hydrographic survey. Especial diligence should be employed to locate and get the minimum depth of water over all reefs, isolated rocks or other dangerous obstructions; this very important investigation may require the aid of pilots or other river men who have had occasion to know of the existence of such menaces. In addition to all these, one or two longitudinal lines along the navigable channel should be run through pools and shoals. On every line individual soundings should be made by the leadsman as rapidly as is convenient. The probable error of the depths determined by such a survey should not exceed 5 percent.

It is impracticable, as well as unnecessary, to exactly locate every sounding taken at ordinary depths. With the sounding boat moving on reasonably straight lines and at a fairly uniform speed, as it should, the two, three or four soundings made between the located ones may be interpolated without material error. Of course, the speed of the boat must not be so great as to cause undesirably large spaces between located soundings, nor is it economy to have the rate of prog-

ress too slow; but usually the exigencies of the work will naturally require a moderate speed which results in a suitable distribution of soundings, as too slow a rate prevents sufficient headway to keep a good line against the varying current effects, and too great a speed interferes with the leadsman's operations. Several located soundings are desirable on every cross-line as a minimum; and although the spacing between the individual soundings on a transverse line is much closer than that between lines, this result is precisely what is needed as irregularities in depth are thus disclosed in approximately equal degree both transversely and longitudinally, because the influence of the current greatly elongates such irregularities in the direction of the greater spacing between lines.

25. The Instrumental Location of Soundings.—Soundings in rivers should always be located by some positive method, and not be entrusted to "sounding on a range," "rowing at a uniform speed," "sounding at regular time-intervals," or other apparently correct but actually suppositious and self-deceiving method of intended location which is incapable of detecting the drifting or irregular progress of the sounding boat caused by such disturbing influences as the varying wind and current, even when the propelling mechanism and the control of the steersman is assumed to be perfect. The endeavor to secure straight lines at a uniform speed is always desirable in order to reduce errors to a minimum, even when positive instrumental location is made; but the contention is that this desirable control of the boat's course should not be implicitly relied upon for indicating its actual position at any instant. The true location of hydrographic points should be as much a matter of definite and complete instrumental observation as is the same procedure in topographic surveying. No engineer would think of running a profile across a valley on a line defined only by ranges on the ridge, when he is to certify to its correctness as the profile along a definite line; still less, because of the persistent influences of the irregular currents and winds on the unstable boat, should he consent to draw a straight line from shore to shore on the located range, when the map of the survey is made, and plot the soundings on this line unless he has instrumental evidence that the intended path has been actually traversed in taking the soundings. Any lingering doubt of the wisdom of discarding methods of location involving assumptions that may cause substantial errors should be set at rest when it is stated that the cost of a hydrographic survey will be no

greater when it is properly planned to make use of definite instrumental location.

There are three standard methods of locating soundings in river surveys: by intersection of two azimuth lines observed from two transits on shore, by the use of two sextants in the boat or of one transit on shore and one sextant in the sounding boat, and by stadia. The first is described in all important books on hydrographic surveying, the second is treated in most of them and the third is discussed in such standard books as Johnson and Smith's "Principles and Practice of Surveying," seventeenth edition, pp. 341–42, and Patton's "Treatise on Civil Engineering" (1903), pp. 1517–18. In all these methods the identity of the locating and sounding observations should be verified by recording the exact time of each, in each of the three field books; and in the first two methods involving azimuth lines from a transit on shore a signal is made from the sounding boat, at the instant that the lead-line is vertical, in order to make simultaneous location observations upon the sounding being taken, while in the third method, the signal for the identification of location and its corresponding sounding is given at the transit on shore. The accuracy of any of the three methods is about equal to that of a topographic survey. A typical survey (made in 1889) of Tillman's Bar, $5\frac{1}{2}$ miles above the mouth of the Ocmulgee River, is shown in Fig. 14,[1] on which the positions of the located soundings are indicated by black circles; the intermediate soundings are omitted from the figure. Although the velocity of current here was very slight and no wind was blowing during the hydrographic survey of this bar, the irregularities in the lines of soundings (which were intended to be perfectly straight and equally spaced) indicate the errors that would have escaped detection if positive methods of location had not been substituted for the frequent assumption of "sounding on a range."

The first method mentioned is the most generally adaptable of any. It may be employed on any river from the largest to the smallest, although its use on the latter is often more or less hampered by the fact that the short bends, narrow stream and overhanging trees and other vegetation make necessary the frequent moving of an observer, as the sounding advances, in order to keep the boat visible from both instruments all the time, thus delaying progress. Another minor difficulty with this method on the smaller streams is the fact that the time signals for locations, as made for the tran-

[1] From Ex. Doc. No. 215, H. R., 51st Congress, 1st Session.

sitmen, are usually one minute apart; and this locates hardly enough soundings on the short, transverse lines. This objection may be avoided by making half-minute signals and locations, but such rapidity of observation is unusual. This method requires the services of two transitmen for making the location observations. A hydrographic survey of this type is described in the Report of the Chief of Engineers, U. S. A., 1888, p. 1110.

The second method is also capable of use on all rivers, with the same occasional practical difficulties mentioned in the last preceding

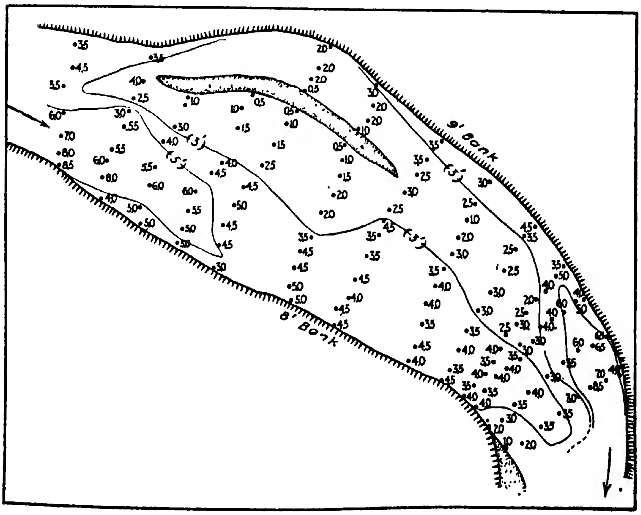


FIG. 14.—Irregularities in lines of soundings.

paragraph. It is possible in this method to omit the services of one instrument man whenever the duties of the chief of the party will allow him to operate a sextant in the sounding boat. Whether both instrument men should occupy the sounding boat, using sextants, or one of these observers should be given a transit on shore to observe an intersecting azimuth line for location, is a matter of convenience; in the first case the plotting of the locations is much more tedious than in the latter, while in the second case a relatively less important disadvantage is the fact that the field work is somewhat delayed by the interruption of soundings while the transitman is changing his station. In the case of using a transit on shore, this

instrument station should form one of the points sighted by the sextant from the boat in order not to complicate the plotting. Sextant locations were used on nearly 80 percent of the sounded length of the river in the survey of the Mississippi.

The third method referred to is hardly practicable on the largest rivers because of the limiting distance at which the stadia can be advantageously used. But on hydrographic surveys of most streams, the cost is somewhat less because only one instrumental observer is needed; the progress is somewhat more rapid than that made under the first method because work is suspended only while one observer changes station, instead of two changing alternately; and there are no vague locations, because the polar coördinates of the stadia method always fix a point definitely, while the intersection of the azimuth lines involved in the other two methods gives rather doubtful results when these lines approach parallelism, as they sometimes will. The principal difficulty attending its employment probably arises from doubt of the capacity of the transitman to make the observations for both azimuth and distance so nearly instantaneously as accuracy requires; but this difficulty is rather imaginary than real, as expertness comes with experience. If the observer will never clamp the upper motion but will acquire that delicacy of touch which keeps the vertical hair on the leadsman until the lateral movement is stopped by releasing the touch at the instant that the lead is vertical; and if he will keep the lower stadia hair on a foot mark of the rod which is about as high above the water surface as is his telescope, when the boat is near, in order to require only slight movement of his gradienter tangent screw, then these difficulties will disappear as experience is gained. Locations being made entirely by one instrument man, they may be as frequent as his rapidity of observation will allow. There are cases on record of more than a thousand located soundings in one day's work, a rate of more than two per minute. The author used this method throughout the survey of the upper Tennessee River (188 miles) in 1891, upon which about 170,000 soundings were taken during the nineteen weeks of field work[1] which cost about $11,000 including the work of the two topographical and two level parties on shore. On the survey of the 1980 miles of the Mississippi River from Minneapolis to the Gulf of Mexico, the location of soundings by sextant observations was abandoned in 1895; and thereafter, on

[1] Report, Chief of Engineers, U. S. A., 1893, p. 2336.

the remaining 400 miles, the stadia method was adopted[1] after checking its accuracy during the first season by stationing a transitman at each end of the line of soundings "the sum of (whose) readings should of course be constant and equal to the distance between the instruments;" for some reason azimuths were not read on this work but reliance was placed on keeping the boat on range.

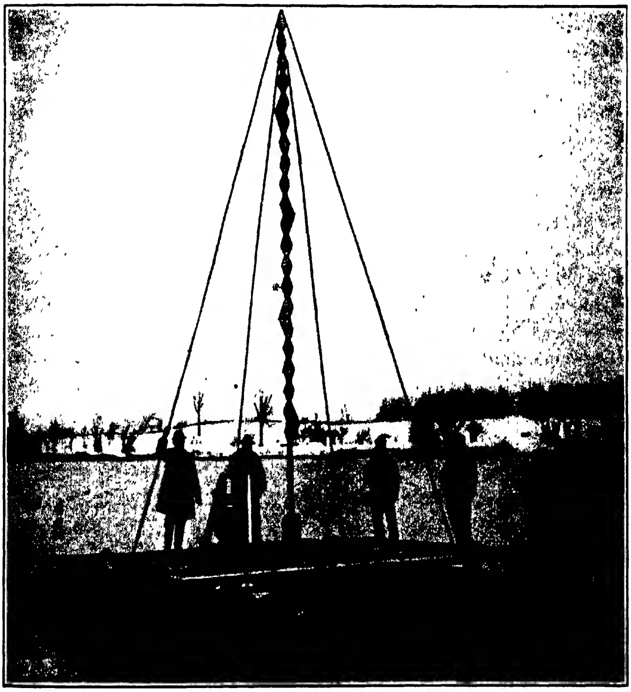


FIG. 15.—Sounding catamaran, with stadia.

In 1899 a survey of 136 miles of the middle Tennessee River was made, on which the soundings were located by the stadia method;[2] the field work occupied about eight weeks and cost about $28 per mile including hydrography, topography and leveling; the office work entailed an expense of about half that of the field work. The

[1] Report, Chief of Engineers, U. S. A., 1896, p. 3520.
[2] Report, Chief of Engineers, U. S. A., 1902, p. 1805.

same method was used on the resurvey of the St. Lawrence River[1] in 1900–02, locations by stadia and azimuth being practicable as far as 4200 ft. from the observer. Two transitmen alternated in locating soundings, one observing while the other was moving to his new position, thus avoiding delays. Soundings were made at intervals of twenty seconds, every third one being instrumentally located. Apparently more than eighty thousand soundings were made on this work, covering about 66 square miles. The cost is not given, but it is stated that "The advantages of this method in both the field and office work are apparent." A view of the sounding catamaran with stadia in position, as used on this survey, is shown in Fig. 15 (p. 101).[2] On the hydrographic survey of the Danube River the Austrian government has used the complete method of locating soundings by the "reading of stadia and horizontal angle."[3]

In the case of some relatively small rivers on which the nature of the traffic and the character of the needed improvements were such that no topographical surveys were required, and where no local magnetic disturbances exist, hydrographic surveys have been made which were based upon compass bearings for direction and stadia readings for distance. On four surveys of this sort, made on Georgia rivers in 1889, surveys in detail of all shoal places were made by running both transverse, longitudinal and channel lines about 100 ft. apart, on which soundings were located by stadia, and by azimuth whose zero was in each case the magnetic north. In all other portions of the river, only the channel line was sounded, and these soundings were given positions as assumed by estimated distance from the bank and by the supposed uniform rate of rowing between the transit stations on the river bank; they should have been located instrumentally. From the located instrument stations the banks and meanderings of the river were sketched. Such surveys are essentially of a "preliminary" character, as far as their scope is concerned; but the relative economy and rate of progress, which they allow, mark them as a type which is advantageous in special cases. On the survey of the Ocmulgee River, five and one-half weeks were required for completing the field work of the 203 miles of length,[4] the cost of which was about $6 per mile.

[1] Report, Chief of Engineers, U. S. A., 1903, p. 2763.
[2] From Report, Chief of Engineers, U. S. A., 1902, p. 2774.
[3] Engineering News, 1903, Vol. 50, p. 67.
[4] Report, Chief of Engineers, U. S. A., 1890, pp. 1458–84.

Occasionally, soundings in winter are made through ice, where the climate is severe enough to solidly freeze the surface of the river. Such methods have been used in the surveys of the Illinois River, the St. Louis River, Minnesota, etc., as well as on surveys of lakes and bays. A standard method of procedure, with costs, is described in the Report of the Chief of Engineers, U. S. A., 1903, p. 1796.

26. Reduction of Soundings to Low Water Stage, for Mapping.—All river improvements are planned with especial regard to low water conditions; for then the more usual difficulties, minimum volume and minimum depth, manifest their greatest limitations. As such works are constructed for the purpose of accommodating traffic in boats of maximum assumed dimensions, the most general and fundamental specification of river improvement projects is the "minimum depth at low water." This fact has a direct bearing upon the determination of the time at which the general survey should be made; because the lower the stream may be, at the time of the hydrographic field work, the more closely will the results indicate the conditions that will exist at the low water stage. As this extreme occurs only occasionally the survey cannot await its advent; but advantage should always be taken of the ordinary low water season which annually occurs on practically all rivers.

The soundings made at any time give the depths with reference to the water surface existing at that time; and the obvious procedure would seem to include the determination of the vertical distance separating the standard low water surface from the water surface existing at the time of the survey, so that this difference might be applied as a correction to the measured depths in order to show the conditions characteristic of the low water stage. There are, however, various facts which complicate this apparently desirable operation.

In the first place the procedure just indicated assumes that the river bed, in its absolute position, remains unaffected by the changes in the volume of flow; while experience has shown that the assumption is generally not true, but that the processes of erosion and deposition now lower and now raise the elevation of any part of the river bed, through the influence of the varying velocities changing with the fluctuations in the stage of the stream. If the material composing the river bed is firm and resisting, the proposed method of correction involves little error from this cause; but in the case of streams carrying a heavy load of silt and sediment or flowing over alluvial soil, the assumed procedure would lead to serious misrepre-

sentation of the low water depths. For example, cases have not been rare on surveys of the Mississippi River where the sounded depths over bars were less than the stage of the river, indicating that the top of the shoal at the time was higher than the low water surface. In fact, so uncertain is the proposition, when applied to unstable streams, that no attempt is made to reduce the soundings in alluvial rivers to equivalent depths at low water; but rather it is the custom to plot the depths as determined by the survey, and indicate on the map the stage of the river which existed at that time. Judgment, experience, and the indications of possible surveys of the same bar at lower stages of the river must guide the engineer in reasoning from ascertained facts to expected low water conditions, and the lower the stream at the time of the survey, the closer to the truth will be this estimate; but such reasoning, however capable, would not justify an attempt to apply such proposed corrections to measured depths so as to plot them as though true at the low water stage, in the case of an alluvial river.

In the second case, where the river bed is of such character that it is approximately stable, the vital error of this procedure is removed. The remaining difficulties are, however, serious. The stream is either gradually rising or falling as the survey proceeds, and so the surface of reference is constantly changing; but this condition can be adequately met by a thorough system of temporary gauge readings always kept at the exact locality where the sounding is being done, as the work progresses, thus giving one term of the final correction. But the particularly difficult operation is the reduction from any stage to that at low water, and it results from the fact that the two water surfaces are not parallel to each other; but even when the river is stationary, they exhibit a progressively varying distance apart, which is increasing or diminishing as one passes down-stream over a shoal or through a pool. This characteristic is well indicated in Fig. 16[1] representing a profile of a portion of the Great Kanawha River, on which the flood water surface is shown to have an almost uniform slope, while the low water surface is very irregular, being nearly horizontal through the pools and sloping abruptly over the shoals. Probably the best way of dealing with this situation is to place permanent bench marks at least at the head and foot of every shoal place, with which the water surface at the time of the survey is connected by level readings. When the stream is at its lowest stage of the season, before the mapping is done, a level party must

[1] From Report, Chief of Engineers, U.S.A., 1899, p. 2484.

rapidly traverse the river and again take level readings connecting this lower water surface at all these points with the same bench marks. There will thus be determined the vertical distance between these two surfaces, at the head and foot of each shoal, which has occurred during the survey. By referring to corresponding readings on the permanent river gauges, and knowing their low water mark, a ratio can be established by whose use the approximate extreme low water can be computed at each bench mark established for this purpose. This will not give exact results because the ratio factor is not constant; but the error is not great in rivers of approximately stable bed if the stream is reasonably low at the time of the survey, and especially if it is near the extreme low water stage at the time of the second series of observations. An example of the use of this method

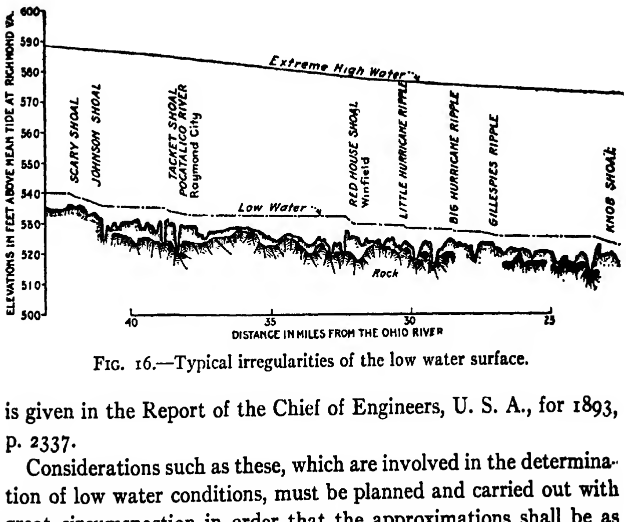


FIG. 16.—Typical irregularities of the low water surface.

is given in the Report of the Chief of Engineers, U. S. A., for 1893, p. 2337.

Considerations such as these, which are involved in the determination of low water conditions, must be planned and carried out with great circumspection in order that the approximations shall be as close to the truth as possible. Of course the occasion for such a procedure is disappearing as time passes, giving opportunity for low water observations or for re-surveys at this stage.

The results obtained by the topographic and hydrographic surveys are combined in mapping. There are generally made three classes of drawings: an "index map" on a single sheet, necessarily drawn to a small scale, usually between 1:60000 to 1:500000, in order to show in outline the whole course of the stream; a considerable number

of "general drawings," on a scale which in some cases is as large as 1:2400 and in others as small as 1:20000, but which is often about an average of these values, giving successive sections of the river with its bank lines, soundings, contours and cadastral features to above the flood line, and other similar information; and sheets of "detail drawings," one of each shoal or bar, at a large scale which varies on different surveys from 1:1000 to 1:4000, so that the depths and general conditions of these particularly difficult portions of the river can be shown with great completeness and definiteness for facilitating the especial study which such places require. These, with a channel profile of the river drawn to a convenient scale, constitute the usual maps which are needed.

27. River Gauges and Gaugings, Current Observations, Etc.—In addition to the soundings, there are necessary the definite determination of all possible authentic extreme low water and high water marks through the valley, and the careful observation of all changes in height of river during the progress of the work; it is also generally necessary that the hydrographic survey establish permanent river gauges and make observations for velocity and volume of flow. These staff gauges are most conveniently established at the larger towns in order to make sure of permanent records; but it is particularly desirable to have them situated near the mouths of the large tributaries, in connection with the gauging stations.

The volumes and velocities are obtained from the gaugings of the stream, which should be made at least in the vicinity of the entrance of all large branches. The complete current meter method of gauging is preferable, but the judicious employment of rod floats is allowable. Both methods are described in standard text-books on hydrographic surveying.

The object of the gauging is not alone the determination of velocities and volumes at the stage of river existing at the time of the survey. Although very important when they are secured at a low water season, these observations should be but one of a series of such gaugings that are made at different heights of water varying from the lowest to the highest, as opportunity shall occur, so that finally a complete discharge curve may be drawn for each established gauging station. It is very important that those in authority should be constantly prepared to have a gauging made whenever the stream reaches a stage for which an observation is wanting; this is particularly necessary at very low or very high water. After once securing such discharge curves and the auxiliary cross-sections

and velocity diagrams, it is only necessary to know the stage of the river, from the readings of the permanent gauges, in order to determine the approximate volume and velocity values at any time and place.

Usually a properly constituted party can finish the field work of a single gauging of a stream at one station in a day, or less. The computations require perhaps an equal amount of work, and the costs are small compared to the value of information obtained. Some of the principal difficulties and errors attending discharge observations are discussed in Trans. Am. Soc. C. E., Vol. 34,

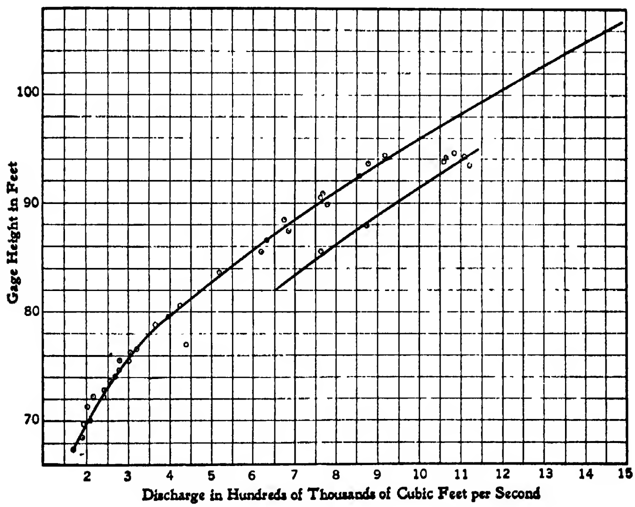


FIG. 17.—Discharge curves, Mississippi River.

pp. 363–377. The accuracy of the gauging depends, of course, upon conditions and the details of the method employed; but results should usually be within 10 percent of the truth, and skillful planning and execution may make the gauging values reliable within 5 percent. It might seem that the employment of discharge curves would be equally reliable, but often this inference would not be warranted. Errors of considerable magnitude may result from dependence upon the discharge curves of an alluvial river, because of changes in the cross-section and velocity of the stream either at the gauging station or at the position of the permanent

gauge; while an additional error of noticeable magnitude is inherent in all discharge curves, due to the fact that the volume of flow of a stream is greater when any certain reading is reached on a rising stage than when the same reading occurs on a falling river. This phenomenon is illustrated in Fig. 17 (p. 107), taken from the Transactions of the American Society of Civil Engineers, Vol. 34, p. 465, where the shorter discharge curve of the rising river gives volumes of from about 20 to 30 percent in excess of those occurring at similar stages when the river is falling. Of course this difference in discharge is not the same for different streams, nor for the same stream under differing circumstances such as varying rapidity of rise or fall; but the fact has been quite widely noted in the most careful gaugings of rivers, and logically leads to the proposition that a discharge hydrograph should consist of a zone whose limiting curves are those of the rising and falling stream, instead of the single, average curve now customarily drawn. This figure also illustrates the fact that the zero of gauges is often not at the low water surface; in this case the gauge reading at low water is 58.

Occasionally a civil engineer is obliged to estimate minimum or maximum volume when there has been opportunity to secure only a few gaugings. If this is unavoidable, it is generally best to make the approximation through one of three independent processes. The first consists in drawing a smooth curve in both directions through the few plotted points which represent the gaugings that have thus far been obtained; this method should carry the greatest weight if the gaugings represented in this tentative discharge curve cover a rather considerable range of volume. The second procedure is also graphic, and utilizes the average or typical discharge curve (which Debauve says is a parabola convex to the axis of gauge heights) by drawing such a curve through the two or three plotted discharge values that have been obtained, and extending this parabola as needed. The third proposition is the familiar "rainfall-run-off" method, which is far too uncertain to be given much credit, particularly in a large river, but which cannot be entirely ignored when an estimate is forced by circumstances. Great care and good judgment are needed in arriving at each of the three values, particularly in the case of the last; and also in weighting each one, according to its probable dependability, in arriving at the final estimate. Even so, the result must be strictly regarded as only an estimate which is liable to many times the error of a discharge

curve, and therefore not available where definite values are essential. For example, in case of an improvement requiring a certain minimum low water flow, if such estimate gave a volume 70 percent greater than that needed, it would probably be safe to proceed; but if the estimate were only 15 percent greater, the execution of the project without more exact information certainly would be hazardous.

Such estimates of the volume of flow, and even the gaugings of streams, are becoming less necessary as the various government bureaus extend such work and complete the records. Volumes and velocities are vital factors in the planning of many river improvement works; and, when not so, their importance in various ways renders them a necessary part of the hydrographic survey.

Another type of investigation, which is an essential procedure antecedent to the planning of any project involving the regulation of a river, is the determination of the characteristics of its currents at different stages, and especially at low water. Nothing is more fundamentally misleading than the fiction that flow takes place in a filamental fashion, that the threads of current-flow actually lie parallel to each other and to the axis of the stream. The very form which the flowing water gives to the bed absolutely refutes such a conception. The influence of the curvature, form and condition of a river channel upon the character of the currents being recognized, there must be realized the necessity of determining them before any works of regulation can be intelligently or adequately planned. They constitute the molding agency whose influence is to be directed by the works of improvement in a way to ameliorate the navigable channel; and therefore their local characteristics must be apprehended and thoroughly understood both at, and for a considerable distance above, the place to be regulated. The investigation is probably best made by submerged floats (sometimes called "subsurface," or "double" floats). That especial type, illustrated in Fig. 18 (p. 110), is preferred because the lower part can be set to any depth at which the course of the current is wanted and thus minimize the influence upon its movement of the wind, and also of the upper currents of the river which are often not parallel to the deeper ones, especially in the vicinity of shoals. The average course and velocity of a float can readily be determined by timed locations made by any of the standard methods of definitely locating soundings.

In addition to the usual hydrographic operations described, special observations are sometimes necessary, such as velocity

observations where considerable fall occurs in a short distance; the volume flowing behind islands, and so lost to the channel; etc.

This discussion, of course, does not consider the instrumental work and surveys needed during construction, because their extent and character depend upon the particular nature of each project, and they can best receive any necessary notice in connection with the descriptions of typical river improvements. Such instrumental work is based on the surveys herein outlined, which are primarily required for the adequate planning of the needed works of improvement as a whole.

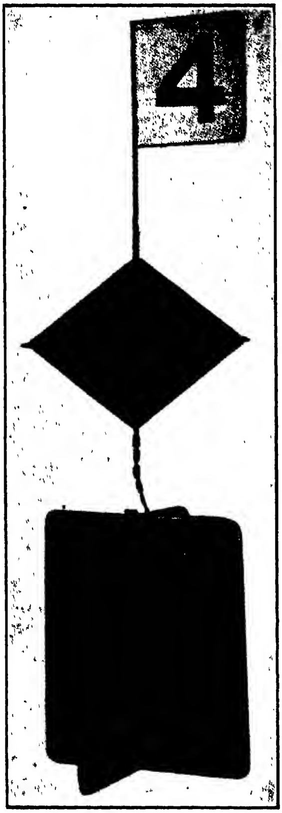


FIG. 18.—Submerged float.

28. Federal Control of Navigable Streams.—The constitution of the United States gives to the federal government the control of the navigable waters of the country. Section 8 of Article I contains the following:

"The congress shall have power,

"1. To lay and collect taxes, duties, imposts, and excises, to pay the debts and provide for the common defense and general welfare of the United States; but all duties, imposts, and excises, shall be uniform throughout the United States. . .

"3. To regulate commerce with foreign nations, and among the several states, and with the Indian tribes."

Story's "Commentaries on the Constitution of the United States" interprets and discusses the definite significance of these two paragraphs, as follows:

"Sec. 908.

"The former opinion has been maintained by some minds of great ingenuity, and liberality of views. The latter has been the generally

received sense of the nation, and seems supported by reasoning at once solid and impregnable. The reading, therefore, which will be maintained in these commentaries, is that which makes the latter words a qualification of the former; and this will be best illustrated by supplying the words, which are necessarily to be understood in this interpretation. They will then stand thus: 'The congress shall have power to lay and collect taxes, duties, imposts, and excises, *in order* to pay the debts, and to provide for the common defense and general welfare of the United States;' that is, for the purpose of paying the public debts, and providing for the common defense and general welfare of the United States. In this sense, congress has not an unlimited power of taxation; but it is limited to specific objects—the payment of the public debts, and providing for the common defense and general welfare. A tax, therefore, laid by congress for neither of these objects, would be unconstitutional, as an excess of its legislative authority. In what manner this is to be ascertained, or decided, will be considered hereafter. At present, the interpretation of the words only is before us; and the reasoning, by which that already suggested has been vindicated, will now be reviewed.

"Sec. 991.

"In regard to the practice of the government, it has been entirely in conformity to the principles here laid down. Appropriations have been limited by congress to cases falling within the specific powers enumerated in the constitution, whether those powers be construed in their broad or their narrow sense. And in an especial manner appropriations have been made to aid internal improvements of various sorts, in our roads, our navigation, our streams, and other objects of a national character and importance.

"Sec. 1273.

"So far as regards the right to appropriate money to internal improvements generally, the subject has already passed under review in considering the power to lay and collect taxes. The doctrine there contended for, which has been in a great measure borne out by the actual practice of the government, is, that congress may appropriate money, not only to clear obstructions to navigable rivers; to improve harbors; to build breakwaters; to assist navigation; to erect forts, lighthouses, and piers; and for other purposes allied to some of the enumerated powers; but may also appropriate it in aid of canals, roads and other institutions of a similar nature, existing under state authority. The only limitations upon the power are those prescribed by the terms of the constitution, that the objects shall be for the common defense, or the general welfare of the union. The true test is, whether the object be of a local character, and local use; or, whether it be of general benefit to the states. If it be purely local, congress cannot constitutionally appropriate money for the object. But, if the benefit be general, it matters not, whether in point of locality it be in one state, or several; whether it be of large, or of small extent; its

nature and character determine the right, and congress may appropriate money in aid of it; for it is then in a just sense for the general welfare.

"Sec. 1274.

"But it has been contended that the constitution is not confined to mere appropriations of money; but authorizes congress directly to undertake and carry on a system of internal improvements for the general welfare, wherever such improvements fall within the scope of any of the enumerated powers. Congress may not, indeed, engage in such undertakings merely because they are internal improvements for the general welfare, unless they fall within the scope of the enumerated powers. The distinction between this power, and the power of appropriation is, that in the latter, congress may appropriate to any purpose, which is for the common defense or general welfare; but in the former, they can engage in such undertakings only as are means or incidents to its enumerated powers. Congress may, therefore, authorize the making of a canal, as incident to the power to regulate commerce, where such canal may facilitate the intercourse between state and state. They may authorize lighthouses, piers, buoys, and beacons to be built for the purposes of navigation. They may authorize the purchase and building of custom-houses and revenue cutters, and public warehouses, as incident to the power to lay and collect taxes. They may purchase places for public uses; and erect forts, arsenals, dock-yards, navy-yards, and magazines, as incident to the power to make war."

It therefore appears that the federal authority over navigable waters rests upon the "paragraph 1" above quoted, as far as the appropriation of funds involved in their regulation is concerned; and upon the "paragraph 3," already quoted, in exercising its administrative control over such waterways, both where appropriations are concerned and where they are not; in fact, to the federal power to "regulate commerce" is generally attributed its authority over navigable waters. Examples of the latter are the complete and exclusive authority of the federal government over such structures as dams, bridges and bridge piers (to high water line) over navigable streams, approval of the general plans of which must be secured from the Secretary of War in all cases, permission for construction usually requiring a special act of Congress;[1] and even after erection such structures are in general subject to federal orders and control, as stated below.

Congress has occasionally declared a certain stream to be navigable. The United States Supreme Court has defined the term in the following words:

[1] See provisions of River and Harbor Act approved March 3, 1899.

"Those waters must be regarded as public navigable rivers in law which are navigable in fact. And they are navigable in fact when they are used, or are susceptible of being used, in their ordinary condition, as highways for commerce over which trade and travel are or may be conducted in the customary modes of trade and travel on water."[1]

A stream of sufficient capacity to float logs or timber to market has been held to be navigable in fact.[2] And even though a river before its improvement had contained obstructions to an unbroken navigation, consisting of rapids and falls, yet, inasmuch as a large interstate commerce was successfully carried on over it in large vessels drawn by animal power, it was held by the Supreme Court to be navigable in fact.[3]

29. Limitations of Riparian Ownership.—Riparian owners of the banks of navigable streams have absolute title only to high water mark; in some states (*e.g.*, Iowa) the title beyond this line rests in the State; in others, the line of division is low water mark (*e.g.*, Missouri); and in others (*e.g.*, Illinois), private ownership extends to the "middle line of the main channel of the river." As clearly stated by the Supreme Court,[4] "The later judgments of this court clearly establish that the title and rights of riparian littoral proprietors in the soil below high water mark of navigable waters are governed by the local laws of the several States, subject, of course, to the rights granted to the United States by the Constitution." And, "the right of the riparian owner, where the waters are above the influence of the tide, will be limited according to the law of the State, either to low or high water mark, or will extend to the middle of the stream."

Whether the title below high water mark is vested in private parties or in the State, the ownership is subject to the predominant authority of the federal government over the navigable waters of the United States. A recent decision of the Supreme Court states that

"This title of the owner of fast land upon the shore of a navigable river to the bed of the river is, at best, a qualified one. It is a title which inheres in the ownership of the shore; and, unless reserved or excluded by implication, passed with it as a shadow follows a substance, although capable of distinct ownership. It is subordinate to the public right of navigation, and . . . is of no avail against the exercise of the great and absolute

[1] 10 Wall., p. 563 (1870).
[2] 8 Bissell, p. 334 (1878).
[3] 20 Wall., p. 434 (1874)
[4] 152 U. S., 40 (1894) and 137 U. S., 669 (1891).

power of Congress over the improvement of navigable rivers . . . . . If, in the judgment of Congress, the use of the bottom of the river is proper for the purpose of placing therein structures in aid of navigation, it is not thereby taking private property for a public use, for the owner's title was in its very nature subject to that use in the interest of public navigation. If its judgment be that structures placed in the river and upon such submerged land are an obstruction or hindrance to the proper use of the river for purposes of navigation, it may require their removal and forbid the use of the bed of the river by the owner in any way which, in its judgment, is injurious to the dominant right of navigation. So also, it may permit the construction and maintenance of tunnels under, or bridges over the river, and may require the removal of every such structure placed there with or without its license, the element of contract out of the way, which it shall require to be removed or altered as an obstruction to navigation. . . . So unfettered is this control of Congress over navigable streams of the country that its judgment as to whether a construction in or over such a river is or is not an obstacle and a hindrance to navigation is conclusive."[1]

The same decision makes clear that Congress has definite control of the flow and of the water power capabilities of navigable streams:

"So much of the zone covered by this declaration as consisted of fast land upon the banks of the river, or in islands which were private property, is, of course, to be paid for (by the federal government taking it). But the flow of the stream was in no sense private property, and there is no room for a judicial review of the judgment of Congress that the flow of the river is not in excess of any possible need of navigation, or for a determination that, if in excess, the riparian owners had any private property right in such excess which must be paid for if they have been excluded from the use of the same . . . It is at best not clear how the Chandler-Dunbar Company can be heard to object to the selling (by the government) of any excess of water power which may result from the construction of such controlling or remedial works as shall be found advisable for the improvement of navigation, inasmuch as it had no property right in the river which has been 'taken.' It has therefore no interest whether the government permit the excess of power to go to waste or be made the means of producing some return upon the great expenditure" for improvements at Sault Ste. Marie.

The United States Supreme Court has also affirmed the absence of federal responsibility to private parties for the inundation of their lands resulting from the increased height of flood waters caused by the building of levees, basing its decision upon the fundamental principle that

[1] 229 U. S., 53 (1913).

"the plenary power of the United States to legislate for the benefit of navigation and to construct such works as are appropriate to that end, without liability for remote or consequential damages, has been so often decided as to cause the subject not to be open." Nor does the reconstruction of a levee on a line farther from the river, thus exposing lands to overlow which had previously been protected, make the government liable for losses resulting from its new location, because of the principle previously referred to; the new position of the main embankment being in consequence of the natural changes in local conditions.[1]

[1] 230 U.S., pp. 1-35, (1913).

CHAPTER IV

METHODS OF RIVER IMPROVEMENT

30. The Preliminary Requirements of Navigation.—There are few rivers of the world which are extensively navigable without improvement. The most notable examples of such exceptions are the Amazon, navigable for 2700 miles for steamers drawing 14 ft.; the Yangtse, 615 miles at 9 ft. and 980 miles at 6 ft.; and the Mississippi, 295 miles at 30 ft. In its primitive condition a river generally presents certain difficulties and dangers to navigation, the removal of which constitutes the preliminary work of making it a commercial highway of the region traversed. The most usual and evident of such adverse conditions, encountered in pioneer navigation enterprises, are logs, trees and snags, rock points and isolated reefs, and uncertainty of the channel location.

Logs occasionally become stranded in ways to menace the navigation of the river; trees that have fallen into the stream often become lodged in its waters and form troublesome impediments; and snags, which are the trunks of trees lacking the most of their limbs but having enough of the roots to cause that end to reach the bottom while the trunk floats at the surface, constitute real dangers to river traffic. Such obstructions not only made navigation a serious proposition at all times, but rendered it so hazardous, except by daylight, that boats were unable to run at night until the river was cleared. A half century ago some of our rivers were still impassable in places, due to the accumulation of such floating débris at points where channel conditions were liable to induce their massing into great aggregations to which was given the name "rafts." The Reports of the Chief of Engineers of the early seventies give lengthy accounts of what was probably the greatest and most troublesome of these obstructions. Its location was in the Red River near the Arkansas boundary; its existence had long prevented the navigation of the river beyond this point, and at least one serious attempt to destroy it had failed, in 1844. Finally a specially equipped boat was purchased and outfitted, and a channel was opened through it in 1873, although much work was necessary in succeeding years to

keep open and improve the channel conditions in the vicinity. This Red River raft, at the time of its removal, occupied about 7 miles of the river channel, covering a total area of nearly 300 acres. The general appearance of a portion of it is shown in Fig. 19. The mass of gradually decaying logs, trees and snags not only formed an interlocking aggregation which received fresh accessions from the river above, but it so obstructed the channel, in places even to the bottom, that the retarded silt-bearing current deposited much of this alluvium in and about the raft; and the decaying logs with the depositing sediment furnished opportunity for a vigorous growth of

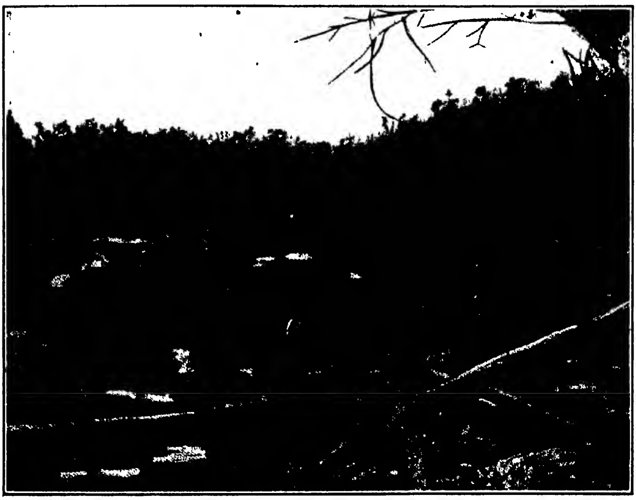


FIG. 19.—The Red River Raft, 1872.

vines, weeds, willows and other small trees over a considerable part of the surface. Besides barring the navigation of the river, this great obstruction had the serious effect of causing overflows and the formation of numerous partial channels in such irregular order, sequence and position that many years were required for the river to recover a natural regimen through this region. The removal of all snags, logs and trees from a river is accomplished by especially designed snag boats, as illustrated in Fig. 20 (p. 118); and the minimizing of their occurrence and of the expense of their removal is secured by the cutting of trees on eroding banks and removing them before

they fall into the river, any overhanging trees being drawn back onto land by block and tackle as they are cut. While all this procedure is very necessary in the first years of navigation, trees and snags collect but slowly once they have been cleared of these obstructions, and a single snag boat can then keep clear many hundreds of miles of river channel.

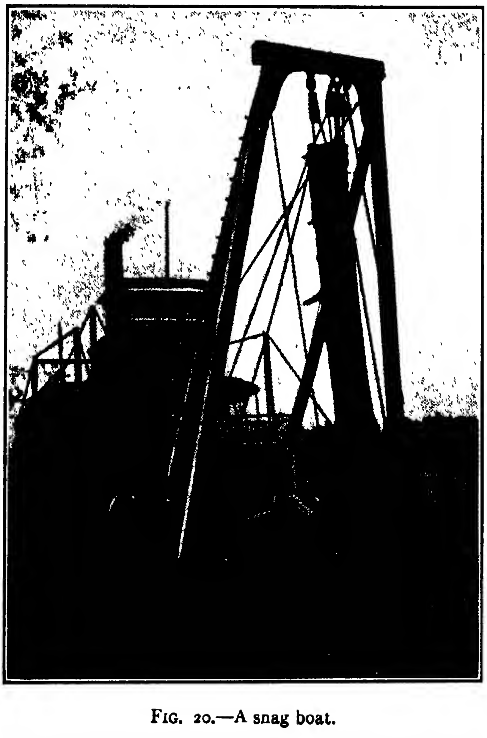


FIG. 20.—A snag boat.

Occasional reefs or points of rock form very serious dangers where they occur; and in the early history of waterways, navigation interests found it advisable to join together in the removal of particularly serious obstructions of this sort, a procedure which soon became a governmental function.

The marking of the navigable channel, because of its uncertainty in many places, has been a charge upon interested governments from

the beginning of systematic navigation; and so sailing ranges are maintained, consisting of beacons whose framework is easily visible by day and on which lights are maintained at night.

While the foregoing operations are very important to the interests of navigation, both financially and scientifically they occupy a very subordinate place. Compared to the proposition of securing reliable channels of depths required for modern commercial needs, the performances already mentioned are insignificant in cost and involve no especially technical qualifications on the part of those who successfully accomplish them; while the improvement of a stream to develop those navigable conditions that are required to secure economic transportation by water involves great expense and engineering ability of particular capacity and insight. Consequently, in discussing the improvement of rivers, it is the following considerations which generally receive all the attention.

31. The Five Methods Employed for the Improvement of Rivers.—Excluding the discussion of those portions of some rivers which are subject to tidal action and other sea influences in their lower portions (which, because of these especial conditions would naturally be discussed in a separate volume on "Tidal Rivers and Harbors," where such improvements are properly classed), there are five ways in which transportation facilities by water, through a river valley, may be distinctly improved: 1, by regulation of the river flowing through it (sometimes termed regularization, normalization, or standardization); 2, by dredging; 3, by its canalization; 4, by the construction of a lateral canal; and 5, by storage reservoirs. The first three methods make direct use of the stream for purposes of navigation; while the fourth is distinctly an artificial waterway throughout its length; and the fifth is comparatively infrequent, as described in Chapter II, its use so far being prominent on only the upper Mississippi, Ottawa, Rhine, Volga, Oder, Elbe and Weser Rivers as auxiliary to their works of regulation or canalization.

A lateral canal has the distinctive features of any canal, viz., a succession of artificially controlled pools of water, each one of which has an almost level surface, the passage of boats from one level to another being secured by means of a lock. Of course, some considerations involved in canal construction are usually simplified in that of the lateral canal, as, *e.g.*, those of minimized earthwork and of water supply for the canal, while others, such as protection from flooding at high water, are more complex; but such matters of detail do not affect its real character as a canal.

So, too, is a canalized river essentially a canal in principle as it also (in so far as its works of canalization are concerned) presents the same distinctive characteristics mentioned in the last preceding paragraph, the succession of pools of very slight slope with the abrupt, vertical change of level at the locks. The particular difference in this case is the fact that the river itself furnishes the location for the slack water navigation artificially secured by constructing dams across the river at the location of the locks, instead of these pools being formed by excavation and embankment as is usual with canals. This again is a matter of detail and in no way alters those distinctively fundamental characteristics which classify it with canals, viz., a slow current in pools at different levels connected by vertical lifts at the locks, instead of the contrasting condition pertaining to rivers, that of sloping water surfaces only. For these reasons both canalized rivers and lateral canals would properly be discussed in another volume of the series, that on "Canalized Rivers and Canals."

Dredging is considered by some authorities as a special form of regulation. It is true that this way of deepening, widening, or straightening channels is very often employed in connection with works of regulation, as well as in maintaining the depth of lateral canals and canalized rivers; but in all these cases it is generally only an auxiliary to those permanent works of improvement. In recent years the efficiency of dredging equipment and operations has been so greatly increased that it is more economical, on some rivers, to incur the periodic expense of dredging channels across obstructing bars than it would be to build the adequate permanent works required to secure the desired low water depths. In many cases the recurring dredging operations are the predominant feature of the improvement, and therefore dredging will be classed as one of the methods of improving rivers, although it is essentially temporary in character.

Works of regulation are not planned, as are those involved in canalization, to radically change the conditions of flow. On the contrary, regulation trains the river toward a greater uniformity of slope, cross-section and velocity than naturally exists, through the direct influence of controlling structures such as training walls, groynes and sills, and of works of bank protection consisting of revetment or spurs, and with the indirect assistance of such auxiliary works as levees, to the extent to which they affect the river bed. The purpose of regulation is, then, to induce rivers to develop to-

ward stability and regularity of regimen—a condition which greatly facilitates their navigability.

32. Lengths and Depths of Important Rivers Improved by Each Method.—Although some nations devoted considerable attention to the improvement of interior waterway routes in ancient times, it was of a very elementary character previous to the invention of locks, in the fourteenth century, which first enabled boats to surmount considerable changes of elevation. However, the scientific development of the canalization of rivers has been a gradual evolution of less than a hundred years, and that of river regulation began about the middle of the last century, in Germany and France, especially. Study and experience have progressively disclosed the principles that may be successfully applied to the regulation of rivers, until this method of improving navigation has become a standard one for attaining the desired effects. Among important rivers of this country[1] on which regulating works have been the predominant feature of the improvement, are the Rappahannock for 106 miles of length, giving a navigable depth of 7½ ft. at low water; the James, 104 miles for a 16-ft. depth; the Savannah, 202 miles, 3 ft.; the Altamaha, 131 miles, 2 ft.; the Ocmulgee, 202 miles, 2½ ft.; the Oconee, 145 miles, 2 ft.; the Suwanee, 135 miles, 4 ft.; the Flint, 105 miles, 3½ ft.; the Chattahoochee, 223 miles, 4 ft.; the Alabama, 312 miles, 5 and 3 ft.; the Pearl, 246 miles, 3 ft. on the lower half and less above; the Red, 800 miles, 3 ft. on the lower half and 2 ft. in the middle part; the Big Sunflower, 180 miles, 3 ft. in the lower part and 2 ft. above; the Arkansas, 463 miles, 3 and 2 ft.; the Black, 239 miles, 3 and 1½ ft.; the Tennessee, 652 miles, 3, 2, and 1½ ft.; the Missouri, 2285 miles, 4 ft. below Kansas City, 392 miles, thence 3½ ft. to Sioux City, and on the remainder, 3 ft.; the Snake, 146 miles, 3½ ft.; and the Sacramento, 262 miles, 7, 4, and 3 ft. In Europe, the Rhone below Lyons, 205 miles, 4 ft.; the Rhine through Germany, 435 miles, having a minimum depth of 10 ft. from the Dutch frontier to Cologne, 110 miles; thence 8¼ ft. to St. Goar, 82 miles; thence 6½ ft. to Mannheim, 79 miles; thence 4 ft. to Strassburg, 84 miles; and thence 3 ft. to Basel, 80 miles; and its two principal mouths, the Waal, 52 miles, 10 ft. and the Yssel, 79 miles, 10 ft. through Holland; the Elbe, 386 miles, 10, 6, 4½ and 3½ ft. through Germany; the Oder, 303 miles, 9, 5, 4 and 3 ft.; the Worthe, 215 miles, 3⅓ ft.; the Weser,

[1] A considerable part of these statistics is taken from the extensive table on pages 61 to 115 of Part I of "Transportation by Water in the United States" (1909), by the Commissioner of Corporations.

227 miles, 10, 5, 4, and 3 ft.; the Vistula, 176 miles, 5 ft.; the Dneiper, 1427 miles, 2 ft.; the Don, 876 miles, 1½ ft.; the Danube, 1400 miles, 10 and 6½ ft.; the Tisza, 431 miles; the Save, 371 miles; and the Drave. 122 miles have been improved principally by regulation.

Canalization has been adopted for such rivers as the Cape Fear River, 145 miles to a navigable depth of 8 ft.; the Coosa, 198 miles, 4 ft.; the Tombigbee, 341 miles, 6 ft.; the Warrior, 140 miles, 6 ft.; the Ouachita and Black, 360 miles, 6½ ft.; the Cumberland, 518 miles, 6 ft.; the Green and Barren, 180 miles, 5 ft.; the Kentucky, 240 miles, 5½ ft.; the Muskingum, 91 miles, 6 ft.; the Great Kanawha, 90 miles, 6 ft.; the Monongahela, 131 miles, 7 ft.; and the Illinois, 225 miles, 7 ft., in the United States. Abroad, among the most important canalized rivers are the Marne, 114 miles, 7 ft.; the Saone, 232 miles, 8 ft.; the Salle, 90 miles, 4½ ft.; and the Moskva, 112 miles, 4 ft.

Works of improvement have taken the form of lateral canals in the valleys of the Loire, 160 miles; Garonne, 120 miles; and the Meuse through Holland 80 miles, although its improvement by regulation also is now being accomplished in order to serve a more extensive territory. However, lateral canals are now rarely constructed except for comparatively short distances, in passing rapids, falls, or other serious obstructions in rivers, as exemplified by portions of the lower Tennessee River at Colbert Shoals and Muscle Shoals, the Ohio at Louisville, the Columbia at the Dalles and the Cascades, and the first cataract of the Nile, at Assouan, which will allow, when lateral canals are built around five other cataracts, a continuous navigation of 2900 miles at a minimum depth of 4 ft.

During the last decade dredging has become a conspicuous feature in the work of maintaining the required navigable depth of the lower Mississippi; and it is the predominant characteristic on some of the great rivers of Europe, such as the Volga, 1600 miles to depths of [illegible], 4, 3 and 2 ft.; the Oka, 840 miles, 2 ft. minimum; the Kama, 796 miles, 2 ft.; and the Po, 337 miles, 7 ft. In this country dredging has constituted the particular method of improvement used also on the Roanoke, 129 miles, 10 ft. on the lower half, and 4 ft. on the remainder; the Pamlico and Tar, 108 miles, 9, 4 and 2½ ft.; the Waccamaw, 97 miles, 12, 6 and 2 ft.; the Great Pedee, 167 miles, 9 and 3½ ft.; the St. Johns, 276 miles, 23, 13 and 5 ft.; Bayou Lafourche, 107 miles, 3 ft.; and the Red River of the North, for a length of 180 miles, securing a navigable low water depth of 2 ft.

33. Rivers Employing More than One System, or Changing the Method.—It must not be understood that the methods of improve-

ment on any river are always of one class or type. Often a stream exhibits such different characteristics in various parts of its course that one method is advantageous in one portion and another method in another part, especially if it be a large river. Marked changes in regimen are indications of possible advantage in change in the character of the improvement which should be planned. Thus, the Seine, canalized above Rouen to a depth of 10½ ft. for the 140 miles to Paris and to 6½ ft. for 61 miles further, is improved by regulation to a depth of 20 ft. for the 80 miles betwen Rouen and the sea; the many branches of the Danube in Hungary have such a limited volume of flow that the greater portion of their length must be canalized, instead of applying regulation as on the main river, and the same fact is true for all but the largest branches of the Volga. Typical of the same general situation in this country is the improvement of the White River, the lower 264 miles of which is improved by regulation to a depth of 3 ft.; while the slopes and channel conditions of the 89 miles above Batesville are such as to cause the projected improvements to take the form of canalization. In the Willamette River, the 12 miles below Portland has its channel depth maintained at 25 ft. by dredging, which method is also used to secure the depth of 12 ft. from Portland to Oswego, 8 miles; but on the 111 miles above, regulation constitutes the main reliance in securing the desired channel depth of 3 ft. In Canada, ocean steamships are able to ascend the St. Lawrence River to Quebec, 410 miles from the Gulf of St. Lawrence; from that city to Montreal, 160 miles, dredging is employed to secure a channel depth of 30 ft.; and from the latter city, for the 170 miles to Lake Ontario, lateral canals with locks of a depth of 14 ft. have been constructed around the rapids and falls for an aggregate distance of 110 miles.

The Tennessee has regulating works in its upper 200 miles of channel; a lock and dam (canalization) in its middle portion, 33 miles below Chattanooga; lateral canals at Muscle Shoals and Colbert Shoals where serious rapids occur about 200 miles below Chattanooga; and dredging is employed on its lower portion. The extreme variation in methods of improvement of navigation seems to be exhibited on the Mississippi River, where the storage reservoirs of its headwaters relieve the low water conditions from St. Paul to Lake Pepin; regulation is applied to maintain a 6-ft. depth on the 658 miles above the mouth of the Missouri River with one exception where the Des Moines rapids led to the construction of the old lateral canal, 5 miles in length, at Keokuk; regulation and dredging are

combined to secure a navigable depth of 8 ft. at standard low water on the 210 miles between the mouths of the Missouri and Ohio Rivers; and the prominence of dredging (although regulating works are numerous) is notable from the mouth of the Ohio to that of the Red River in obtaining the required minimum channel depth of 9 ft. through this distance of 750 miles; while the 295 miles from the Red River to the Gulf of Mexico is naturally of ample depth, even for ocean steamships, there being only one bar in all this distance, and that near the upper end, on which the depth at low water is less than 30 ft.

It is also true that many rivers illustrate a change in the method of improvement which has been applied to them, due usually to either an increase in the projected depth, failure of the system used to secure the desired results, or to advancement in knowledge and modes of operations which have made applicable for such cases a method which had before been impracticable. Thus, in deepening the upper Rhine from 2 to $2\frac{1}{2}$ meters, it was found that additional regulation would not also secure the desired "width and gradient suited to the heavy traffic navigation" on the reach between Bingerbrück and Assmanhausen, and so a lock and lateral canal were constructed for the passage around these rapids, while regulation continued to be the method used on the remainder of the river.

The development of improvements on the Oder River is characteristic of German experiences on portions of the Elbe and Main also, as well as upon parts of some rivers of other countries. In 1897 the work had been completed by canalizing the upper portion, from Cosel to the confluence of the River Neisse, and from the latter point downstream, 303 miles, the improvement consisted of works of regulation. However, experience on the 46 miles from the Neisse to Breslau proved unsatisfactory, as the navigable depth was reduced at times as low as 3 ft. For this reason it was determined in 1905 to canalize this portion also, in order to secure a minimum depth of 8 ft. of water, for the better accommodation of the rapidly growing commerce of this section of the country.

The first method of improvement planned for the Volga, but carried on quite intermittently, was that of regulation. Probably due to that procedure and to the "great fluctuations of water level" in this huge river with its alluvial bed, the works were not as effective, in its lower portion, as had been expected, and now dredging is the characteristic method used. So, too, on the lower Mississippi, the chief present reliance for preserving the navigable depth is dredging.

Transportation by water through the valley of the Mohawk took the form of a lateral canal, located above the flood crests of the river, under the various enlargements of the Erie canal up to the construction of the project calling for 9 ft. of depth. By the adoption of the Barge Canal Project of 1903, under which the depth is being increased to 12 ft., it became increasingly difficult to enlarge the lateral canal on its side hill location; and other difficulties such as loss from percolation, which in the smaller canal amounted to from 1½ to 13 cu. ft. per second per mile of canal when the water supply is limited, led to the abandonment of the lateral canal for the new improvement. Inasmuch as the low water discharge varies from only 254 cu. ft. per second, in the upper portion of the valley considered, to 373 in the lower portion, and the slopes range from 1.31 to 3.55 ft. per mile, averaging 2.26, it would have been impossible to secure by regulation the desired channel prism 200 ft. wide and 12 ft. deep. Therefore the method of improvement has been changed to that of a canalized river.

Perhaps the most notable example of a radical change in the method applied to the improvement of a great river, which has been adopted up to this time, is the abandonment of the regulation of the Ohio River, 967 miles in length from Pittsburgh to its confluence with the Mississippi River; and the complete substitution of the method of canalization under the project of 1908. This will involve the construction of forty-eight new locks and dams at an estimated original cost of $63,731,488 (in addition to about $5,000,000 previously expended on six such dams in the upper 30 miles of river), $800,000 per year for maintenance and $200,000 per year for "the necessity for dredging in the pools." The minimum low water depth is to be 9 ft. throughout the course of the river. It appears that the controlling reason for this fundamental change in method of improvement is that the upper part of the river lacks a sufficient volume of discharge to give the desired width and depth, at the surface slopes existing, by the method of regulation. Since the federal government commenced its improvement by the removal of obstructions, the concentration of flow in a single channel and the general regulation of the river, the requirements of commerce had always been in advance of the channel dimensions obtained. With a low water discharge at times as small as 1200 cu. ft. per second at the head of the river and a natural low water slope in the first 90 miles averaging almost 1 ft. per mile and locally reaching sometimes six to ten times that amount, it is not strange that it was

found impossible to secure a low water depth there of even 3 ft. Consequently five locks and dams had been constructed in the upper 24 miles of the river previous to 1908, the upper one, that at Davis Island, having been completed in 1885. The peculiarly advantageous situation of the river, extending from the great manufacturing and mineral region of western Pennsylvania to the Mississippi River, and so offering the possibilities of transportation by water from Pittsburgh to industrial centers in the Ohio River valley as well as to New Orleans and St. Louis, and beyond, marks its channel as a natural highway of commerce which requires adequate development to accommodate the great and rapidly growing traffic requirements of that part of the country. The extensive use of the river at favorable stages, and especially the economic advantage of transporting commercial products in fleets of barges as already discussed in Chapter I, led to the adoption of the nine-foot project as already stated; the locks to be large enough to accommodate the smaller tows, and the movable dams to be of ample width so that, when lowered at medium stages of the river, fleets of the larger size may pass through unhindered. Not only has the adopted plan extended the slackwater project over the middle portion of the river, with its low water discharge volumes of 5000 to 8000 cu. ft. per second; but it also involves the canalization of the lower 190 miles, where the available volume of flow is doubled, the slope averages less than 4 in. per mile, and the Ohio here assumes general characteristics similar to those of the middle and lower Mississippi River, except that the bed largely consists of a more stable material. This conclusion was reached after the comparative estimate for improving the Ohio, below the mouth of the Green River, by dredging indicated that its canalization would be only about two-thirds as expensive. No alternative estimate was made for the improvement of the lower Ohio River by regulation. Probably one factor which influenced the adoption of canalization for the lower part of the river is the fact that when the Mississippi River is in flood and the Ohio not so, the backwater effect is felt scores of miles up the latter; and this situation would greatly complicate the effectiveness of regulating works if built.

34. Conditions Favoring the Various Types of Improvement.—The decision as to which form of general improvement should be adopted for any river is a question that requires a very thorough knowledge of the river and its regimen, and also of the commercial facilities to be secured; as well as of the characteristics, capabilities,

adaptability, costs and serviceability of the different methods of river improvement.

In general, regulation is the most desirable kind of permanent improvement of rivers which have a low water discharge sufficient to give the needed dimensions of boating channel; whose slope is not too great (as, for example, average slopes of perhaps a foot or two for the smaller rivers and proportionately less for large rivers, with local slopes limited to about three or four times the general ones); and whose required increase of depth is not too great. The reasons for this general principle are that regulation is the least expensive in first cost and in maintenance of any of the methods involving permanent works; it requires no expenditure for operation; it generally offers a maximum of service and freedom of movement to boats navigating such river; in cold climates there is a minimum of interference by ice; and where there is a surplus volume of low water discharge, future increase of depth may, when desired, be secured by adding to the works if these are designed with possible modifications in view, thus avoiding excessive expense until the full amount of improvement is needed, as has occurred in very many instances; it is capable of modification as experience indicates the need; and it is entirely adaptable to conditions, as one shoal after another may be improved under the general plan in order of their importance, without awaiting the completion of the whole project for the attainment of some temporary relief.

Dredging, as a distinct system of improvement, is usually an alternative to regulation. Its principal advantages are the avoidance of so great an initial outlay for the regulation works, but at the expense of a considerable periodic (usually annual) expenditure for deepening the shoal places by excavation; that, with a sufficient plant, the results of operations are immediate and generally sure; and the further facts, sometimes of great importance, that this method of improvement (because of its very character) does not impose upon the river a fixed condition or method of permanent improvement while the commercial conditions to be served may be in process of evolution and hence very indefinite. Being generally an annual operation, the depth and other details of excavation are entirely responsive to changing requirements, with no reconstruction whatever being necessary. This method seems generally advantageous on very large alluvial rivers on which regulation works would be so expensive that the alternative annual cost of the required dredging would prove a financial economy.

The canalization of a river is best whenever the navigable depth required is very considerably greater than that of the natural channel at low water; when its slopes are so great that the resulting velocities seriously impede its navigability or threaten its stability of regimen; or if the minimum volume of flow is so small that, with the slope of surface encountered, either the width or depth of regulated channel will be deficient for the traffic requirements. Canalization is entirely capable of transforming streams of great slope or of comparatively small discharge into navigable waterways; its results are sure and effective as soon as construction is completed; up-stream navigation is made easier and often speedier; and some of the dangers of river navigation are reduced. The formation of the comparatively quiet pools above the dams is often of great economic advantage at commercial centers, offering, as they do, advantageous harbor facilities for industrial purposes. "For such reasons, on the upper Ohio, between Pittsburgh and Beaver, where five dams have already been built at a cost of about $5,000,000, the increase in value of the mere land on one side of the river due to the development of deep slack water pools was much greater than the total cost of all five dams."[1]

These three methods, or a oombination of them, are the usual ones employed in river improvement. Of course, where the expense of land and the construction of dams for storage reservoirs is not too great, these may be planned to supply the deficiency in volume of flow; but such opportunities are relatively rare. Because of their great comparative expense, caused largely by the fact that especially valuable land must be bought for their location, that excavation and construction costs are great, and that their location must be above flood heights if they are to be constantly usable, and also because of other disadvantages compared to open-river navigation, such as reduced speed, cost of operation, etc., lateral canals are unusual except in cases where rapids or falls exist under such local conditions that a short canal at the side of the stream constitutes the most eonomical way to secure adequate facilities for passing such obstructions. Yet instances occasionally occur in which lateral canals cost less and better serve the communities interested than would the alternative of canalization, due to especial topographical conditions and difficulties of regimen of the river.

35. Some General Comparisons of Cost.—In order to form a general idea of the experience of France, Germany, and the United

[1] Final Report of the U. S. National Waterways Commission (1912), p. 200.

States with regard to the first cost of each of the three methods of permanent improvement, it may be stated that the average initial expenditure on more than 2400 miles of regulated rivers, for which the figures are available, was nearly $36,000 per mile; on nearly 2200 miles of canalized river it averaged more than $44,000 per mile; and on nearly 300 miles of the principal lateral canals the cost was more than $150,000 per mile. A similar estimate,[1] involving fourteen rivers, makes the relative cost of canalization about one-third greater than that of regulation. Suggestive in a different way, because they form estimates of cost of different methods for the same river, are those of the Oder at $36,000 for regulation and $87,000 per mile for canalization; and of the Loire at $100,000 per mile for a lateral canal and $20,000 per mile for regulation. In the study of the 182 miles of the Mississippi River from St. Louis to the mouth of the Ohio River, in accordance with the huge project to secure a channel depth of at least 14 ft. at low water and a minimum width of 500 ft., the estimated expense of complete regulation was $257,000 per mile for first cost and $2700 per year per mile for maintenance, to which is added an estimate of $35,000 per mile for expenditures for dredging to secure and maintain the proposed channel depth during the fifteen years of construction of such regulation works; if dredging alone were relied upon for the method of securing the navigable depth of 14 ft., the first cost of dredging equipment is placed at $33,000 per mile and that of annual operations and maintenance at $11,000 per mile, while no allowance seems to be included for renewal of the dredges when they shall be worn out in service; corresponding figures for canalization are $312,000 per mile first cost, and $1400 per year per mile for maintenance and operation for the "locks and fixed dam" project, and $137,000 per mile first cost and $2100 per mile annual expense for operation and maintenance if movable dams are substituted, while, again, no allowance seems to be made for renewal; for a continuous lateral canal the estimate averaged $559,000 per mile in first cost and $3300 per mile annually for operation and maintenance; storage reservoirs as a means to secure the proposed depth were found to be impracticable. "After considering the advantages and defects of all the methods of improvement proposed, the Board concludes that the most practicable method of obtaining and maintaining a navigable channel of 14 ft. depth from St. Louis to Cairo is by the completion of the project of 1881 for partial regularization in such a

[1] Brochure 9 of Twelfth International Congress of Navigation.

way as to secure a permanent controlling depth of 8 ft. (estimated to cost $115,000 per mile), and then to rely upon dredging for securing and maintaining any further increase of depth; the side contraction works to be so located as to be in harmony with further works of improvement by complete regularization, if in future such works be found necessary and advisable,"[1] the expenditure for which plan is estimated to be $135,000 per mile for first cost and $8200 per mile annually for operation and maintenance; apparently the particular reason for this conclusion of the special board in the case of this project of very unusual magnitude is its grave doubt of the economic advantage of a navigable depth of 14 ft.

36. Difficulties and Objections Inherent in Each System.—Referring now to the objectionable features of each of the three methods of improving rivers which are most employed, it may be stated that canalization is ordinarily considerably more expensive than regulation or dredging under similar circumstances; and especially so if the river is very wide necessitating long dams, if its banks are low, thus requiring numerous locks and dams in order to minimize the flooding of valuable valley lands, or if movable dams are constructed to reduce the damage and loss caused by the increased flood height of high waters which generally results if fixed dams are employed. This kind of improvement also involves a very considerable maintenance cost, and a constant expense for operation which is lacking in regulation. There is, further, a delay for lockage at each change of level, for which there is a certain measure of compensation due to the greatly reduced velocity of current at the lower stages against up-stream traffic, resulting from the approximate slack water conditions, but which minimized velocity occasions a loss in time to down-stream traffic theoretically somewhat less than the up-stream advantage. It is also usually necessary to await the completion of the entire system of canalization before receiving any considerable measure of advantage from its inauguration. Whenever conditions require extensive change or reconstruction, as for renewal or to secure increased depth, the consequent work is very expensive. European experience indicates a serious harm to agricultural interests, because the raised surface of the canalized river interferes with the drainage and elevates the ground water level of lands in its vicinity.

Dredging requires unfailing attention, at a very considerable expense, at every low water period; and it is not always practicable

[1] Document No. 50, H. R., 61st Congress, 1st Session.

or possible to deepen all the shoal places as fast as is necessary with the capacity of the equipment provided. Because of the constantly recurring character of necessary operations, there is some danger that exigencies of various sorts (*e.g.*, lack of available funds) may occasionally prevent their prosecution at the time at which they are needed. Further, the added channel depth which may be secured by dredging is often a rather moderate one.

The cost of regulation is considerable, and it is not always readily modified to meet changed commercial requirements. If not skilfully designed and adapted to local conditions, it may fail to secure that degree of minimum depth which should be obtained, or may do so only after considerable modification and delay. The difficulties involved in securing precise effects in the regulation of rivers, subject as they are to large variations in volume of flow and in velocity, and consequently exhibiting such variability in their eroding and transporting power and so involving the fundamental problem of controlling sedimentation, makes impossible a regulated channel whose dimensions shall more than approximate to those desired. The definite control of sediment and silt has proved so uncertain in practice, in many cases, that some engineers have been inclined to assume a skeptical attitude upon the whole question of regulation. One rather extreme instance of this occurs in a paper on "The Limits Attainable in Improving the Navigability of Rivers by Means of Regulation,"[1] in which one of the conclusions is that "Only rivers or long reaches of rivers in which natural erosion is fully developed are adapted to regulation. The navigability of unfinished rivers, yet in a state of erosion, can be improved with permanent results only by canalization." The argument of the paper leading up to this conclusion indicates that the statement is not intended to be applied as a general truth, but rather to the ultimate permanent increase of depth to its approximate theoretical limit, where a reasonable increase in minimum depth by regulation may have already been attained; as is indicated by this preceding statement, "It is easier to make a wild river double its depth than to deepen a regulated river by a few centimeters." And yet the first mentioned statement has been repeatedly quoted, even to the present time, to the confusion of a clear apprehension of the adaptability of works of regulation. A large portion of the rivers which have been improved by this method would have remained unavailable for navigation if the

[1] H. Engels in Transactions, American Society of Civil Engineers, Vol. 29, pp. 202–222, and Vol. 30, pp. 492–497.

quoted "conclusion" had been literally true. The residuum of uncertainty in attaining the ultimate depth possible is the real adverse element which must be allowed for.

Regulation, then, is typically adapted to rivers of fairly stable regimen having a considerable low water discharge and a moderate slope. Canalization better applies to rivers of great slope, small discharge, or to those in which a very considerable added depth is needed. Dredging is favored for alluvial rivers, of a markedly unstable regimen, having a large volume of flow, in which permanent works would be comparatively uneconomical, or when their construction may be questionable because of indeterminate traffic requirements of the future. Lateral canals are particularly advantageous for the passing of serious rapids and falls sometimes found in navigable rivers, and occasionally in other special circumstances. Reservoirs for the storage of water for use at low stages are important in cases in which the volume of water needed to augment the natural flow may be economically obtained in this way. It is the usual attitude to regard regulation as the essentially desirable method of improvement, sometimes assisted by storage reservoirs when necessary and available, and often supplemented by dredging to the extent found expedient; although occasionally the method of dredging has been found so advantageous as to be practically predominant, at least for the present. If for any reason regulation is not feasible, canalization is chosen; and a lateral canal for an extended stretch of river is usually considered a last resort. A general indication of the situation is the fact that, of the 26,400 miles of navigable rivers of this country, only about one-eighth is canalized.

Local conditions, mainly those appertaining to the immediate river and its regimen, but also those of the valley and the region, will form the basis of the study in comparing the relative advantage of the different methods of improvement, and will thus indicate the particular one to select as the main reliance for the accomplishment of the desired purpose. Yet the auxiliary service of other methods should always be considered, and is often advantageous. For decades the German experience has closely associated regulation and dredging in streams of stable bed, the method of regulation being predominant; while in large alluvial portions of rivers the practice has frequently been to deepen primarily by dredging and to maintain these depths by the aid of regulation. An example of the latter case in this country is the lower Tennessee River, on which

the channel has been deepened by dredging; and this is generally found to result in a rather permanent improvement, partly because of the placing of the dredged material so as to guide the water toward the dredged cut, as far as this is practicable. On the upper Tennessee a similar procedure is usual in the firm gravel and rock found in the shoals, producing there entirely permanent works of improvement. Examples of regulation with dredging as auxiliary are numerous, as on the upper Mississippi River. Cases where reservoirs serve to aid regulation have already been cited, as in the Weser and Mississippi; and the former lateral canal along the Mohawk River, as well as the recent canalization of the same stream to serve the enlarged barge (Erie) canal, are both served by storage reservoirs.

CHAPTER V

THE PRINCIPLES OF REGULATION

37. General Consideration of Conditions that are to be Modified.—The fundamental conception governing the design of well-planned works of regulation is the judicious control of stream flow in a way to minimize the natural disparity between its pools and shoals. Rivers would have a very superior degree of navigability if the shoal crossings between the pools of successive concave banks and in comparatively straight reaches might be eliminated; as stated at the International Engineering Congress in 1904:[1]

"The Mississippi River is not as shallow a stream as many persons seem to think. At extreme low water, the depth along most of the distance below Cairo is 40 ft. or more, and it is only over the bar crossings that the water is ever shoal enough to interfere with navigation. The bars that extend across the stream are usually comparatively narrow ridges, and occur where the channel crosses from the end of the concave bank on the one side to the beginning of the concave bank on the other side of the river. If all of the material in these cross shoals were cut away, and deposited in the long deep pools above and below, there would be a depth all the way south of Cairo, of 35 ft.—probably more—at extreme low water. The high water plane, being about 50 ft. above that of low water, would, consequently, make a depth of 85 ft. or more at extreme high water. Of course, we can not expect that this ideal condition will ever be realized, but the tendency is in that direction."

The paragraph just quoted assumes, of course, that the river surfaces would remain unaltered in elevation and slope, a condition which would be hydraulically impossible; but it well illustrates the comparative depth and extent of the pools and shoals. To make a portion of a river to have an approximately uniform width and depth, which would be intermediate between those of the natural pools and shoals, is entirely impracticable of accomplishment because of the enormous cost which would be involved in transforming the natural channel into an artificial one whose longitudinal curves and slopes and whose successive cross-sections would have to be everywhere so extensively modified and hydraulically adjusted as to secure approximate uniformity under a multitude of confusing

[1] From Transactions, American Society of Civil Engineers, Vol. 54, Part D, p. 463.

influences, prominent among which are the persistently disturbing phenomena of erosion, transportation and deposit of sediment; effects which are enormously aggravated and complicated by a great variability in volume of flow. However, it is practicable to considerably reduce such excessive differences in depth and other less vital characteristics, and so to greatly improve the navigability of rivers.

That the proposed regulation must be judiciously planned is indicated both by theory and experience, and especially by the latter. In applying this method of improvement the river's flow is not to be checked, as in canalization, nor otherwise radically interfered with; but rather is its onward movement to be controlled and directed in such a way as to preserve the fairway through those relatively short portions of its course where a dissipation of its energy allows the obstructing shoals to naturally persist. To secure this desired continuity of favorable conditions a persistently skillful guidance, a technically correct control, an expertly designed regulation will accomplish much where a more rigid restraint, defective correlation of characteristics of regimen to desired effects, or excessive constraint will lead to defective or even disastrous results. As expressed by the Twelfth International Congress of Navigation:

"Many and strong natural forces come into play on a large stream; sometimes their power is enormous and their laws are very complex. The struggle engaged against these forces is severe, costly and of doubtful success; failure is certain if the end in view be contrary to the laws which govern them. But if, instead of fighting, no more be done than to lead them; if, instead of trying to change wholly the nature of their effects, we be satisfied with guiding them by gradually changing the direction of their action through a series of concordant efforts which follow each other steadily and are continuously superposed, success is much more sure, because the result to be attained at each point is very small and, therefore, possible and the means at hand are better proportioned to the end."

The significance of a continuously conducive influence in the successful control of the usual sedimentary stream can otherwise be expressed as a proper regard for the fundamental principle of the conservation of energy in the interest of its navigation. For in the planning of its works of regulation the civil engineer not only designs so that the previously dissipated energy of the river is now concentrated upon a certain portion of the obstructing shoal to secure its potent aid in obtaining a continuous channel; but also so that this is effected in the most advantageous portion, which is

identified as that course which causes the least aggregate interference with the natural flow in order that the energy of the river may no longer become so reduced as to allow the bar-forming deposition of sediment to continue; or, as far as the details of design of such regulating works are concerned, it means the practical minimizing of all conditions that produce friction, impact, cross-currents, eddies and all such phenomena which diminish the energy and so reduce the capacity of the stream to maintain the desired channel conditions at all stages.

38. Statement of the Two Different Theories of Control.—There is, then, a thorough agreement of the highest authorities upon the need of a guiding control of a river's flow in a way to constructively direct its channel-forming capabilities; but the practical methods employed to secure that result differ very much, even in principle. Although such works of regulation as planned frequently combine, to a greater or less extent, the application of two different theories, yet fundamentally these distinct principles appear as the basis of design of regulating works, sometimes vaguely and at other times quite definitely. The first system can best be distinguished as that which artifically molds the cross-section of the stream to the required size and shape, where necessary, to secure the desired width and depth, the channel dimensions thus constituting the direct subject of control; and this system has so far constituted the usual basis of design of works of regulation, and is therefore the type which characterizes the greater part of this discussion. The second system, on the contrary, directs the designer's attention primarily to the essential guidance of the current in such a way that its effective energy shall be so utilized as to produce the desired channel conditions; this principle is discussed more at length in Chapter X.

The regulation of a river involves permanent works constructed to either directly or indirectly improve the channel in such a way that the controlled flow of the stream is utilized to maintain its navigability. Works of direct effect are those which control sectional dimensions, and comprise those which laterally contract the river to secure a greater channel depth and those which regulate the elevation of the bottom in order to widen by reducing the depth, to prevent undue scour which may harmfully lower the low water surface, or to secure a reduction in an excessive slope of water surface which would cause an undesirable velocity. Improvement works of indirect effect consist of those which protect the river banks from erosion, a process which contributes the greater part of the sedi-

ment which deposits to form the shoal places; and of those works which, by their guiding influence upon the current, direct its volume in a regular and concentrated channel-forming flow over a constant course, such as training walls or other directing works at only one side of the low water channel, or construction on both banks of the river to keep the direction and volume of the higher waters more nearly coincident with that of the low water channel. Sometimes dredging, levees and reservoirs are included in works of regulation; but their influence involves so different principles that they are given separate consideration in this treatise.

39. Deepening by Lateral Contraction is Based on Hydraulic Formulæ.—The most distinctive feature of regulation as ordinarily planned is contraction in width which, although not always needed, is often required because the most general defect of a natural stream is insufficient depth typically occurring where the wandering currents lead to an excessive width; and so the apparent way to permanently correct this defect is to artificially reduce the width to that which will result in producing the desired depth at the low water stage. As in many other engineering propositions of complex character, in determining the details of design of such a project, reliance should not be placed entirely upon any single procedure; but the indications of theory, observation, experience and experiment should all be united in arriving at the final plans for the improvement.

Hydraulic theory forms the basis of the design of works of lateral contraction. For determining the width that should be given to the channel in order that its average depth may be that which is the essential feature of the project, recourse is had to hydraulic principles as expressed by the well known formulæ, $v = c\sqrt{rs}$ and $Q = av$, in which

Q represents the volume of flow in cubic feet per second,
v represents the average velocity of current in feet per second,
a represents the area of cross-section in square feet,
r represents the hydraulic radius,
s represents the sine of slope of the water surface,
c represents a cœfficient.

By substituting for "a" its components "b" (the average width) and "d" (the average depth), and also considering that "r" is equal to "d" (an assumption which, in dealing with such channels, involves no error of moment), a combination of the two equations results in the relation, $b\sqrt{d^3} = \dfrac{Q}{c\sqrt{s}}$

The first member of this equation contains as factors the variable terms in question. Of the three quantities composing the second member, "Q" is a constant, as the volume of flow of any special stretch of river between tributaries at the low water stage (the condition for which improvements are primarily planned, as it is the least favorable to navigation that can exist) does not materially fluctuate. The factor "s" would be also a constant at this stage if the channel conditions remained unaltered, but as some change in slope generally accompanies the construction of contracting works the usual assumption that "s" is a constant is not exact; still more is it an approximation to consider "c" a constant, because its value is well known to vary with changes both of slope and mean depth, especially with the latter; yet this assumption also is usual and is justified in arriving at the general relation between depth and width of channel for the purposes of preliminary design. Therefore we conclude that the product of the average width and of the square root of the cube of the mean depth of a cross-section is approximately a constant. Other ways in which writers have expressed the same law are that the product of the mean depth and the square of the area of cross-section may be considered invariable; or that the product of the average depth and the two-thirds power of the width is a constant.

While occasionally large error may result from assuming this product of the width and the three-halves power of the average depth to have the "constant" value indicated, as where "s" is considerably changed or the assumed value of "c" is much in error, yet most situations are such that conclusions based on this supposition are reasonably true; and unusual or anomalous conditions, which would lead to substantial error, must be considered as a warning of a corresponding uncertainty in the values obtained from its use under such conditions. So understood, then, that equation mathematically expresses the following principles:

1. A narrowing indicates a deepening of the river and a shoaling is usually accompanied by a widening, whether considered in connection with the comparison of two successive cross-sections of the natural stream or in anticipating the effect of artificial works of improvement.

2. The proportionate contraction is greater than the proportionate deepening, as "b" enters with an exponent of unity while that of "d" is three-halves; thus, if a section of the river is narrowed to half its former width, the assumption gives a resulting mean depth hardly 60 percent greater than before.

3. Each foot of narrowing a river becomes increasingly effective in producing additional depth as the contraction progresses. This fact indicates the need of especial care in such design.

Exceptional conditions under which the above conclusions do not hold good are very considerable changes in the character of the bed and banks; the encountering of a non-erodible material, as a rocky stratum; or of progressive levee construction and timber rafts, as explaining the anomalous case of the Atchafalaya River.[1]

40. Difficulties of Predetermining the Values of the Hydraulic Constants.—In assigning values to the three factors of the second member so that the width, corresponding to any assumed depth (or *vice versa*), can be computed, it is customary to assign to "Q" the measured low water discharge of the stream at the place in question; or, as it is often the case that the regulating works are not water tight, such fractional part of this low water discharge is taken as will make due allowance for the loss from leakage.

The numerical value usually given to "s" is practically that existing before the works of improvement are constructed. Considered with reference to its mathematical accuracy this procedure is incorrect, because of the complex influences of contraction which directly tend to raise the water surface where the contraction occurs, and so decrease the surface slope above and increase it below, and indirectly tend (through the increased velocity and consequent scouring effect) to ultimately deepen the channel until the opposite result to that just given may be produced. Sometimes one influence will predominate and sometimes the other. An estimate as to which condition may prevail can be made by using Chezy's formula (already given) for deriving "v"; computing approximately the equivalent bottom velocity by the formula of Darcy (bottom velocity $= v - 11\sqrt{rs}$) or of Dupuit $\left(\text{bottom velocity} = \dfrac{v}{1 + 0.0073\,r}\right)$; and then, by referring to Table No. 3 (pp. 140–1),[2] find whether the material composing the bed is likely to be eroded. If it is an eroding velocity, the slope through and below the contraction will become progressively lessened and that above similarly increased (unless artificially prevented) through the effect of this erosion of the bed until the slope has so much reduced that the velocity becomes no longer an eroding one for the material composing the river bed. Evidently

[1] Transactions, American Society of Civil Engineers, Vol. 58, p. 1.

[2] From Transactions, American Society of Civil Engineers, Vol. 36, pp. 308–309: "The Suspension of Solids in Flowing Water," by E. H. Hooker.

TABLE NO. 3.—GIVING VELOCITIES OF CURRENT AT WHICH DRAGGING BEGINS

thority	Dubuat	Telford	Blackwell	Sainjon	Login	Rhine measurements	Zschokke	Verein Hütte	Bouniceau
ference	Hydraulique, Paris, 1786, p. 94	Partiot in Annales des Ponts et Ch., 1871, I, p. 34	Proc. Inst. Civil Engineers, Vol. 82, pp. 47-50	Partiot in Annales des Ponts et Ch., 1871, I, p. 33.	Stevenson's Canal and River Engineering, p. 315	Zeitschrift des A. und I. Vereins zu Hannover, 1884, p. 176	Lectures, Zurich, 1895	Ingenieurs Taschenbuch	"Etudes sur la navigation," 1845, p. 19
Remarks	Bottom velocity, feet per second	Bottom (?) velocity, feet per second	Feet per second	Bottom velocity, feet per second	Bottom velocity, feet per second	Feet per second	Feet per second	Bottom velocity, feet per second	Bottom velocity, feet per second
t earth		0.25							
ck clay (allowed to settle ½ hr. in water)					0.250				0.26
ter's clay	0.27							0.26	
t clay		0.50						0.52	0.49
sh, water sand					0.667				
ge sand	0.71								0.72
getable soil					0.833				
d		1.00							0.98
m sand								1.02	
sand					1.103				
avel (size of anise seed)	0.36								0.36
avel (size of peas)	0.62				2.000	2.46 (if disturbed)	0.62		
avel (size of beans)						2.95 (if disturbed)	1.066		
avel (diameter .008 ft.)				0.82					
avel (diameter .03 ft.)				1.64					
ckbat (contents 2.59 cu. in.)			1.75–2.00						
avel		2.00							
lites (2.39 cu. in.)			2.00–2.25						
te (2.38 cu. in.)			2.00–2.25						
pebbles (1.06 in. diam.)	2.13							2.30	2.13
ckbats (4.76 cu. in.)			2.25–2.50						
nts (1.96 cu. in.)			2.50–2.75						
ckbats (18.5 cu. in.)			2.75–3.00						
lites (17.68 cu. in.)			2.75–3.00						
te (9.06 cu. in.)			2.75–3.00						

TABLE NO. 3.—GIVING VELOCITIES OF CURRENT AT WHICH DRAGGING BEGINS. (*Continued.*)

Authority	Dubuat	Telford	Blackwell	Sainjon	Login	Rhine measurements	Zschokke	Verein Hütte	Bouniceau
Reference	Hydraulique, Paris, 1786, p. 94	Partiot in Annales des Ponts et Ch., 1871, I, p. 34	Proc. Inst. Civil Engineers, Vol. 82, pp. 47–50	Partiot in Annales des Ponts et Ch., 1871, I, p. 33	Stevenson's Canal and River Engineering, p. 315	Zeitschrift des A. und I. Vereins zu Hannover, 1884, p. 176	Lectures, Zürich, 1895	Ingenieurs Taschenbuch	"Etudes sur la Navigation," 1845, p. 19
Remarks	Bottom velocity, feet per second	Bottom (?) velocity, feet per second	Feet per second	Bottom velocity, feet per second	Bottom velocity, feet per second	Feet per second	Feet per second	Bottom velocity, feet per second	Bottom velocity, feet per second
Flints (10.37 cu. in.)			3.00–3.25						
Pebbles		3.00							3.28
Boulders								3.08	
Gravel (.085 ft. diameter)							3 09		
Gravel (.13 ft. diameter)				3.28					
Gravel (size, hazel to walnut)						3.48 (if disturbed)			
Gravel (size, pigeon's egg)						3.67 (if disturbed)			
Smallest gravel						3.87			
Gravel (walnut size)						4.92			
Gravel (.17 ft. diameter)							5.15		
Gravel (weight, 250 grams)						4.92 (if disturbed)			
Broken stone		4.00							3.94
Gravel (weight, 2500 grams)						5.90 (if disturbed)			
Conglomerate (soft schist)		5.00						4.90	5.00
Gravel (.33 ft. diameter)				4.92					
Laminated rock		6.00						6.00	6.00
Gravel (.56 ft. diameter)				6.56					
All gravel						6.56 (if disturbed)			
Gravel (.18 cu. ft.)							7.22		
Gravel (1.25 ft. diameter)				9.84					
Hard rock		10.00						10.36	9.84
Gravel (1.09 cu. ft.)							10.29		
Gravel (2.20 ft. diameter)				13.12					
Gravel (2.18 cu. ft.)							15.44		

such a condition not only would make quite uncertain the results of a computed relation between a proposed navigable depth and the corresponding width to give the channel, but also might endanger the contracting works by undermining them; it may also so lower the surface of the pool above that shoals may develop where there was previously a sufficient depth. Any of these unfortunate results, and especially the latter, have too often occurred; as in the case of the Rhone at La Mulatiere, where the low water surface is said to have lowered more than 4½ ft., due to the deepened channel through the shoals below. On the contrary, if the bottom velocity is insufficient to erode the material of the bed at the site of the proposed contraction works, it is necessary to excavate the material to the desired depth of channel, which will then hold itself free from subsequent shoaling if the scouring velocity of current through it is great enough to prevent deposit. The latter case, that requiring an initial excavation of the channel, is a common experience in river improvement; and either this, or the effect of too great a velocity as previously mentioned, is a condition that must be expected to some extent at least, as the ideal "v" for the place is rarely attainable. However, the change in the value of "s" generally is not very great, especially if the contracted portion is relatively long or if comparative stability of slope is adequately planned, and as it appears in the form of the square root, a numerical error in "s" is correspondingly reduced in its effect on the value of the equation.

As for "c," the mathematical part of its determination consists in the use of such formulæ as the well-known ones of Bazin and Kutter. In connection with such computed values it is always wise to refer also to values of "c" actually determined by observation on rivers of similar regimen, condition and dimensions to that under consideration, attributing to this coefficient finally a value which seems most probable in view of these various indications of both theory and observation. In addition to the 121 values contained in the valuable tables of gaugings in Hering and Trautwine's translation of Ganguillet and Kutter's "General Formula for the Uniform Flow of Water in Rivers and Other Channels" (1889), pp. 190-223, Table No. 4 gives some additional and more recent data.

41. The Computed Results are only Approximate.—Actual values of the second member vary, in the case of regulated rivers, from less than 1000 to perhaps 20,000, and occasionally to several times the latter quantity. In any case when its value is determined it is neces-

sary to equate $b\sqrt{d^3}$ to it, substitute the desired value of "d," and then compute the corresponding value of "b." For each shoal place whose permanent improvement is planned, the width so found constitutes approximately that which should be given to the pro-

TABLE No. 4

"c"	q (cu ft. per sec.)	r (feet)	s	v (feet per sec.)	area (square feet)	n	River
64.5	......	3.30	0.00027	2.00	......	0.032	Wisconsin
44.8	3,075	1.87	0.00167	2.50	1,230	0.035	French Broad, near mouth
67.0	11,150	10.05	0.00008	1.87	5,950	0.034	Tennessee, at Knoxville
66.0	17,560	7.75	0.00015	2.21	7,940	0.034	Tennessee, at Chattanooga
80.4	240,000	25.74	0.00015	4.82	49,750	0.034	Tennessee, at Chattanooga
60.0	18,500	8.42	0.00060	1.38	13,400	0.034	Tennessee at Colbert Shoals
88.8	10,250	4.27	0.00015	2.22	4,610	0.030	Tennessee at Riverton

posed navigable channel to which the whole low water flow (as far as it is practicable) is to be confined. Often such computed width cannot safely be taken as final, partly because of the greater or less degree of approximation unavoidable in its derivation, but mainly for the following reasons.

The formulæ involved are based upon the assumption of rectilinear or filamental flow parallel to the axis of the channel, which is usually correct in a generalized way but which may be considerably in error in such places as shoal crossings, due to local circumstances. A striking illustration of this is shown in Fig. 21, (p. 144), which represents a model,[1] of a shoal situated about fifty miles below Kieff, on the Dnieper River; the position of the filaments, trailing from their supporting sections, shows the direction of the current as observed in the river at each corresponding point, and their lengths indicate velocities; the lack of parallelism of many of the filaments, to any axis of the stream which can be assumed, is very pronounced.

The use of "c" in the formula to determine the value of $b\sqrt{d^3}$ involves the conception of the mean velocity of the Chezy formula. This "v" is an average of all the velocities of the cross-section of the stream; and, while the resulting average velocity of the designed channel will be approximately correct and hence the area of cross-section also, there is no assurance that the distribution of the varying velocities of the section will be such that the channel will maintain

[1] From the Fifth Brochure, Sixth Communication, First Section of the Tenth International Congress of Navigation.

even approximately equal depths from side to side, nor that successive sections of the contracted channel will be similar in form, both of which are desirable for its navigation. This conclusion follows from the fact that the mean velocity gives no clue as to whether it is the average of components having a comparatively small or a relatively large variation, nor to the distribution of such variations in velocity throughout the cross-section. If the velocities

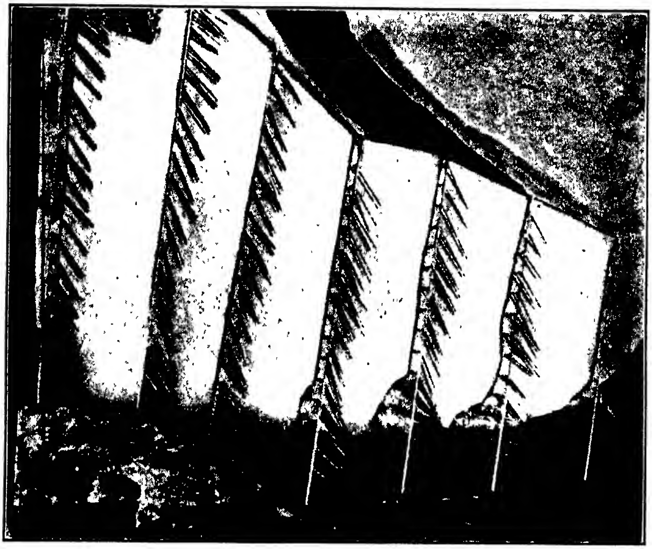


FIG. 21.—Non-parallelism of river currents.

approach uniformity from side to side, the channel will approximate to a uniform depth, and this might be a natural supposition from the theoretical reasoning; but if the velocity near one side of such channel were considerably greater than that at the other, there would necessarily result a greater depth at that side, and a corresponding deficiency in depth at the other. Nor is there any certainty that a shallow portion will continue at the same side, as slight and even undetectable causes (not appearing in the mathematical discussion) may deflect the strength of the current from side to side.

That such irregularities in section may occur in a channel whose design is mathematically correct is thus evident; and the explanation is introduced here in order to caution against either too confiding a

reliance upon theory alone, or too sweeping a denunciation of mathematical results; for the fact is that many channels as improved by regulation have been disappointing not so much in failing to offer sufficient depth somewhere as in not furnishing that approach to uniformity and axial continuity of depth which navigation interests require. Sometimes the contracted channel has had such an excessive depth in a portion of its section that the navigable width was deficient; but more often a slight excess and accompanying deficiency of depth will occur in such a way that a sufficient width of deep water exists at all sections, but the position of the slight shoals is so irregular that the boating channel is too tortuous to be serviceable. When such results occur, either the regulation works must be modified or recourse be had to dredging. It might seem practicable to plan the works of contraction for such an excess of narrowing and deepening beyond the theoretical amount that, even if the uncertainties mentioned did unite to reduce the expected advantages, there still would exist the minimum depth required by the project. There are, however, several objections to this possible procedure; where the volume of flow is limited, that proposition will sometimes contract the width of channel more than traffic requirements will permit; the liability of erosion is always increased, with the attendant dangers already mentioned; in fact, these and other such experiences have led Dutch engineers to state that "The limitation of these widths (of channel) ought to be made with circumspection."

The computed width to give a contracted channel requires, then, either corroboration or a cautious modification; and where the final determination of the proper width is uncertain the works should be so constructed that they may be readily modified to meet the requirements as difficulties may develop.

The principal corroborative or corrective information concerning the right contracted width of channel to adopt is often furnished by observation and study of the same stream whose transverse dimensions are naturally adjusted to an approach to equilibrium of conditions as they locally exist, but which have not been reduced to mathematical formulary. Sometimes the regimen of another unimproved river is so approximate to that of the one whose regulation is proposed that considerable consideration may be given it in deducing the probable effect of the project in hand. Interspersed among the deep and shallow portions of a river, investigation will disclose sections where the stable depth approximates more or less closely to that planned for the improved channel; and a judicious

interpretation and adaptation of all such indications to the projected works should aid materially in reaching a correct conclusion. On this point a most forceful exposition is offered in a report by M. Girardon to the Sixth Congress of Inland Navigation (1894) as follows:

"It seems, first of all, that confidence in hydraulic formulæ is singularly shaken. Without denying the value which these formulæ may have as the result of very exact experiments carried out on a large scale, most engineers have had to record the various disappointments which have been the result of their extension and their use under circumstances differing too much from those under which they had been determined. They give results which are more or less exact and which are, as a rule, sufficient for all practical purposes when it is a question of water alone. But our rivers are not only water courses as they are commonly called; they discharge together water and solid matter, and it is just the movement of the solid materials which causes all the difficulties with which we have to struggle, and it is taking the problem by the wrong end when an endeavor is made to solve it by regulating the flow of the water without seeking at the same time to regulate the movement of the solid materials. And this way of attacking the problem becomes all the worse because these two movements react on each other and because the effects produced on a river with a shifting bottom are, very frequently, not only different from but often contrary to those anticipated and which would have been realized if only the flow of the water on a solid body had alone to be considered. The consequence of this statement is that there is but one sure guide, direct observation of the facts produced under conditions analogous to those in which action must be taken, not in artificial canals wherein water alone passes, but on the rivers themselves where the phenomena which must be considered really occur."

42. Examples of Necessary Supplementary Studies of Rivers.—If contraction works have already been constructed on the same river, in portions of similar regimen either above or below, their effects are the most reliable indication possible; or if the effects of the improvement of another river of similar regimen have been carefully studied, the significant features of the modified channel so produced are also very pertinent.

Unfortunately the range of such thorough technical investigations is as yet rather limited. A very extended study of this kind was made on a portion of the Garonne, about 14 miles in length, which had been regulated a score of years before by works of contraction and bank protection. The most important facts and conclusions[1] of this

[1] Fargue's "La Forme du Lit des Rivieres a Fond Mobile," 1908.

classic research well illustrate the significance and value of such investigations.

The width of the continuously contracted channel normally varied from 560 to 620 ft., but was occasionally more than 800 ft. The average slope was about 15 in. per mile and varied locally from about one-third of this value at long pools to nearly three times that average at some of the shoal crossings, the latter occupying about one-fourth of the total length. The mean discharge, averaged for twenty-six years, was 24,261 cu. ft. per second; the flood flow was nearly six times that amount; and the low water discharge was 3062 cu. ft. per second. No affluents of any importance entered in that portion of the Garonne under consideration. The bed of the river consisted of sand and gravel, the average size of the latter being about that used in road metalling and its occurrence predominating at the head of the banks; the general proportion of sand averaged nearly twice that of the gravel, and its occurrence characterized the lower portion of the banks.

The plan of the works of regulation followed in the improvement of this river not only involved the contraction to the average width of 590 ft., but also consisted in purposely giving to this channel a longitudinal form typically consisting of a succession of curves of varying curvature, such as would best fit the natural channel and at the same time conform to the best practice as then understood; as stated in the introduction, "In the absence of theory, which at present is silent, it is necessary to enquire of nature; that is, to study the curvings of navigable rivers and to imitate them in those parts which have the best channels."

The works having been thus constructed and in existence for so considerable a time that the effects of the changed conditions would have become complete, the detailed study was made in order to determine as definitely as possible the relation between the characteristic features of the improvements and their effects on this particular stream. Fortunately for the thoroughness of the endeavor, very definite records had been kept since 1839 of all important facts, including yearly soundings of quite complete character. The particular questions whose solution was sought were as to whether the artificial banks should be straight or curved, and the variation in effects produced by changes of curvature.

For this purpose the portion of the Garonne was divided into seventeen parabolic arcs which approximately fitted the axis of the channel. Then longitudinal profiles of the channel were drawn,

upon which were represented at each profile point the exact radius of curvature of each curve, the width of channel, the slope of water surface, slope of river bed and other such facts as were essential to a thorough consideration of the questions involved.

In thus readily showing that the cross-section is a function of the longitudinal plan or shape of the channel and that there is an intimate relation between the depth and the longitudinal curvature of the bank, there occurred an excellent opportunity to more definitely determine the details of these relations as existing in this special case; and further study resulted in the formulation of what were called "the six empirical laws," as follows:

The law of deviation. The deepest and the shoalest points in the channel are below the vertex and the ends of the curves, respectively. On the Garonne the average amount of this down-stream displacement is about one-fourth the length of the curve, but is less than that on the shallower crossings.

The law of greatest depth. The point of maximum depth is the deeper as the curvature at the vertex of the curve is sharper. A thorough mathematical and graphical study of conditions resulted in this equation, which gives the relation between depth and curvature;

$$C = 0.03H^3 - 0.23H^2 + 0.78H - 0.76,$$

where "C" represents the reciprocal of the radius of curvature at the vertex of the curve expressed in kilometers, and "H" is the low water depth of the deepest point of the channel in meters.

The law of the trace. In the interest of both average and maximum depths, the curves should neither be too short nor too long. For the Garonne this length is preferably about $1\frac{1}{3}$ km., instead of perhaps 50 percent greater or less length than that.

The law of the angle. For equal lengths of curve, the average depth of the pool is greater as the central angle subtended by the curve is the larger. The equation derived from this analysis is $h = 1.50 (1 + \sqrt{c^2 + 1.71c})$, in which "$h$" represents the average depth of pool and "c" the average curvature in metric units.

The law of continuity. The longitudinal channel profile shows gradual variations only when the curvature changes gradually. Abrupt modifications of depth accompany rapid variations in curvature.

The law of the slope of the bed. If the curvature varies continuously, an increasing radius of curvature marks a reducing depth and

an increasing degree of curvature is accompanied by a deepening. The rate of change in depth, which is the slope of the bed, is represented by the equation: $q = 0.1553p + 0.0114p^3$, which was derived from a careful analysis of observed conditions; "q" representing the variation per kilometer of the reciprocals of the gradually changing radii of curvature of the curve expressed in kilometers and "p" expressing the kilometric variation in depth.

The author of the above laws regards it as very probable that they are general as far as the expressed principles are concerned, although the coefficients derived in fixing the mathematical relations as determined for the Garonne River will probably be different for other streams, as also may be the form of some of the equations themselves.

Concerning the question of the longitudinal curvature of the channel, this investigator has considered several types. He is a pronounced advocate of rejecting circular arcs as not furnishing that constantly varying curvature which he believes essential. As stated above, parabolic arcs formed the basis of the investigation of the Garonne; but in other studies the spiral, the sinusoid, and the lemniscate have been used, all of which conform to his expressed essential requirement of gradually changing curvature, and the last of which has been repeatedly used in the planning of contraction works on European rivers.

It is apparent that the principal fundamental consideration, in this study of the Garonne, was the securing of stability of the bed of the river, and of its regimen in general. This feature is, indeed, exceedingly important; but it surely is of at least equal moment to determine the relation between those hydraulic elements which affect a permanent deepening of the shoal places themselves. Indeed, this latter consideration is paramount, as upon it depends the degree in which the improvement of the river is made effective for navigation. Undoubtedly the "laws" mentioned above have a bearing on this question, but their relation to the fundamental problem of increasing the effective depth is not direct, nor are such essential questions elucidated as those of the complex and interdependent relations between the velocity, curvature, other details of plans at places of reverse curvature, and the resulting increment of depth secured at the shoal crossings.

However, some important auxiliary questions affecting the efficiency of the improvement of the Garonne occur in connection with the discussion of changes in the low water slope of the river

produced by the contracting works. This low water surface was depressed an average amount of about 3 ft. and, in places, to an extent of double this quantity in forty-two years. The effect of this gradual change was to develop new shoals, both directly as a consequence of the lowered river surface and indirectly by the induced erosion of existing bars furnishing sediment (amounting to a million cubic feet per mile) to deposit at shoal places lower down-stream and thus bringing the latter into prominence. The general net result finally produced was "an effective amelioration of comparatively small amount," although the originally quite variable low water slope was approximately equalized, which is a result that many consider to be desirable. It was concluded that a preferable project would have omitted continuous contracting works, and instead should have adopted the plan of improving for navigation the sinuous back channels instead of the main part of the river at places where islands and gravel bars caused an excessive width, and especially should have planned to improve by contracting systematically only in the immediate vicinity of shoals; so that the natural slopes of the low water surface would have been less seriously modified, and only the shoals of limiting depth would have been deepened, thus avoiding the disturbance of so great an amount of troublesome detritus as well as making unnecessary so extensive and expensive construction. Six examples of crossings, both deep and stable, are cited, five of which exemplify the contention that the width should be reduced only at the crossings. The sixth, the deepest of all, was wider at the shoal than either above or below; this particular case was explained by the fact that it had an effective variation in its curvature, it being approximately sinusoidal in form; and because of both its exceptional depth and its permanence, it was characterized as the "type-curve."

More than thirty years ago an analysis of the relation between curvature and depths, in the lower part of the Mississippi River was made by trained men of the U. S. Coast and Geodetic Survey. The summarized conclusions are quoted from the Report of the Chief of Engineers, U. S. A., 1883, p. 2300:

"The portion of the river examined lies between a point 4½ miles below Fort Saint Philip and another point near Point Houmas, distant 150¾ miles. We have used the original hydrographic sheets of the Coast and Geodetic Survey, on each of which the soundings are reduced to the lowest stage during the season. This portion of the river receives no considerable tributary and has no important outlet—unless Bonnet Carré

Crevasse be so regarded, which certainly does not influence the form of the river-bed to any great distance.

"The elements of the river from the general means, for the entire distance, $150\frac{3}{4}$ miles, corresponding to an air-line distance of 105 miles, are: mean width, 2337 ft.; mean depth, 60 ft.; mean channel depth, $98\frac{1}{10}$ ft.; mean area of section, 140,000 sq. ft.

"The study has led to several conclusions, some of which are rigidly inductive, while others are inferential and will be so distinguished.

"*Inductions.*—1. Bends, as increasing the depth of the river, offer, on the whole, no advantage. They cause a depression of the bed at the turns and a corresponding elevation at the reversions.

"2. The mean depth of cross-section, and the channel depth, vary with inverse radius of curvature, or, practically, with angles of deflection.

"3. The variation of channel depth with curvature is limited; the gain of depth with deflection ultimates essentially at 23° to 25° for half-mile chords, and within this limit is nearly proportional. The mean depth increases with curvature at about half the rate found for the channel depth, and within the scope of our observations exhibits no limit.

"*Inferences.*—There appears to be a decrease (comparatively small) of river width with distance going up river, and also a decrease with curvature—both irregular, and regarded as inconsiderable as affecting the relations of other variables mentioned under the head of *Inductions.*

"There is a tendency, at reversions, where one centrifugal force overlaps the other, to divide the stream and induce scour near both banks, with central shoaling.

"Since the shallow water at the reversion seems to be the result of loss of energy by the stream, due to its scour in the bend above, it may be inferred that equality of depth is incompatible with uniformity of width and that a forced contraction at reversions becomes necessary to remove the bars."

The above outline of the studies of these rivers indicates an investigation which is needed on many rivers. If this were done, and if the results were made available, they would furnish a most valuable fund of information to aid in planning similar projects. However, it is very rare that all conditions in comparable rivers will be similar, and therefore due allowance must generally be made for differences in those hydraulic elements which are not the same. An example of a relatively simple adaptation is furnished in discussing the proper width to be given the Waal, which had been reduced in width from 360 to 310 m. in 1889, and still gave trouble from the formation of bars in the channel because of too low a velocity allowing the deposit of silt, much of which enters it from the Rhine above. The latter river had been given a practically uniform width

of 340 m. and had held for many years a stable and well-formed bed at the depth desired in the Waal. Gaugings at low water showed that the mean velocity of the Rhine was 1.15 m. per second, and in the Waal, 0.85 m. and as the gradients of the two rivers are practically the same the width of the Waal should therefore be about $(340 \times 0.85) \div 1.15 = 263$ m. This and other considerations have led to the adoption of 260 m. as the width now being given to the Waal, which is expected to end the trouble caused by the formation of obstructing bars.

43. Other Considerations that are Important in Planning the Improvement.—Before discussing the two types of works of lateral contraction it is necessary to consider some conditions which the improved channel must also fulfil in addition to the proposed navigable depth; namely, the minimum width, the maximum velocity (or equivalent slope) and the exact planning of the longitudinal outline of the low water channel with regard to its profile, position, curvature and course.

The minimum width which the improved river may have is purely a transportation matter, depending upon the dimensions of the boats that are adapted to the navigation and traffic conditions to be met. Of course this width should be enough to allow one boat to pass another safely, and varies in practice from about 80 ft. to 120 ft. or more. For example, the minimum bottom width of the Weser above Minden is fixed at 82 ft.; the French Broad, 100 ft.; the upper Tennessee 120 ft.; etc. Whenever this minimum is greater than the computed width, regulation is impracticable and canalization is the recourse, unless this deficiency be so moderate that it is possible to reduce the slope at such points enough so that the volume of flow will suffice; or unless, possibly, storage reservoirs may furnish the deficiency.

The question of the maximum allowable velocity, or the equivalent maximum slope, is one partly of engineering importance and partly of transportation interest. In the latter regard the velocity must be a practicable amount less than that of the river boats so that they may progress up stream through the channel at a speed depending upon the needs of the service; unless occasional particular conditions favor such a concentration of slope, and consequent velocity, that some special provision is made to assist the boat against an otherwise impracticable current. Examples of such artificial aids are found in this country where warping rings are anchored at the bank so that a boat's towline and capstan may be used to assist the ordi-

nary motive machinery. In similar cases on such European rivers as the Seine, Meuse, Rhone, Rhine, Elbe and Main, cables or chains have been laid along the bottom of the channel, which the boat picks up and uses for pulling itself upstream. At the Iron Gates of the Danube a wire cable, several miles in length, has been long used for the same purpose. However, such devices are quite rare, and their occasional use is gradually being displaced by preferable alternatives such as the construction of a lock and dam, or more often by methods of regulation adapted to local conditons in a way to reduce the high velocities, and still more frequently by the employment of boats of greater motive power and effectiveness. The last-mentioned procedure is prevailing on the Danube where powerful boats successfully stem the current reaching a velocity of 16 ft. per second at the Iron Gates. Some suggestive values of surface velocities which do not very seriously impede traffic through improved channels over shoals are 7 and 8 ft. per second on the Tennessee River; maximum velocities of 5 ft. on the Loire, 6½ ft. on the Garonne, and 7½ ft. per second on the Rhone at low water; and a maximum velocity of 6 ft. per second was adopted in planning the improvement of the Dneiper and of the French Broad Rivers. These velocities may be roughly compared with the usual mean velocity by applying Bazin's formula, Maximum Velocity $= (1 + \frac{25.4}{c})\, v$, or a usual empirical rule that the maximum ordinarily is between 20 and 40 percent in excess of the mean velocity; while the Mississippi River Commission has adopted the relation, Surface Velocity = 1.25 (Mean Velocity).

The mathematical consideration of this question, that of the allowable maximum velocity of the current, may be approached by considering the extra time required to steam on the river from one place to another and back, caused by the velocity of the stream. If the distance in miles between the limits of the trip be represented by "l," the speed in miles per hour of the boat or the towboat and barges (as used on the round trip) in still water be indicated by "u" and the average velocity of the current in miles per hour by "v" the time consumed in steaming up stream will be represented by $\frac{l}{u-v}$, and down stream by $\frac{l}{u+v}$; if we subtract from the sum of these two periods an expression for the time which would have been required on the river if it had no current $(2\frac{l}{u})$, the result will be the number of hours of additional sailing time due to the current; and if we divide

this difference by $\frac{2l}{u}$ the quotient will represent the proportional increased time on the river caused by the current velocity. This last expression reduces to the fraction $\frac{v^2}{u^2 - v^2}$. Thus, if the velocity of current is $\frac{1}{10}$ the speed of the boat in still water, the increase of sailing time required is found to be 1 percent; if "v" equals $\frac{3}{10}$ of "u" the additional time is almost 10 percent; when "v" represents $\frac{1}{2}$ the velocity of "u" the time required is one-third greater; if $\frac{7}{10}$ of "u," it is nearly double that which would have been required in still water; when "v" equals $\frac{9}{10}$ of "u" the time on the river would be more than four times as great; and, of course, for "v" equal to "u" the time is infinite. It is thus seen, in a generalized discussion, that the lost time is very little for small values of "v," but increases rapidly as the current velocity becomes relatively considerable, and with an ever increasing ratio as the velocity of the current approaches nearer to the speed of the boat in still water. Now, time lost in transit is a commercial detriment to the carrier by water, as it involves a practically proportional additional expense for fuel, labor, capital charges on equipment, etc. This is easily realized if it be remembered that time so lost reduces the number of trips a boat can make during a season, and hence lessens the earning capacity. On this basis, that of time lost and so increasing the cost of transportation, it is possible to approximately determine that maximum velocity of current which permits of competition with other transportation agencies and allows successful operation under all existing commercial conditions. Considerations of this sort led to the adoption of about 5 ft. per second as the maximum allowable velocity in the improvement of the Loire, and about 6½ ft. per second for the Garonne. It is hardly necessary to state, however, that traffic advantage requires the velocity of current to be as low as river conditions will permit.

The competitive feature of the case is thus accentuated by C. Valentini of Bologna:

"It may be confidently asserted that the gradient should be under the limit beyond which the force required for towing vessels up stream becomes greater than the effort of traction in the case of an equivalent railway train; since otherwise, navigation could no longer compete with the railways. To obtain an idea of the gradient limit value it will be sufficient to consider that, for example, with a gradient of 2⅗ ft. per mile and a mean depth of 8½ ft. the velocity is already 4½ ft. per second, and that if one adds to this the moderate figure of 2 ft. per second as the speed of a vessel ascending the

stream, the total velocity becomes 6½ ft. Now, it is known that, for such a velocity, the resistance offered to traction, by a vessel under ordinary conditions, begins to exceed the corresponding resistance of an equivalent railway train."

The engineering significance of a maximum velocity limit has reference to its effect upon the stability of the river bed and banks. As discussed earlier in this chapter, a maximum velocity low enough to prevent erosion is desirable in order to avoid the expense of protection works (such as are described in Chapter VII) which must be built if the stability of regimen is endangered by an excessive velocity.

The effect upon the longitudinal profile of the more usual works of regulation, especially as constructed in this country, is to modify it but slightly; that is, the greater slopes remain at the shoal crossings where they existed before the improvement; but in case the fall is so rapid that the velocity would seriously impede navigation or cause excessive erosion, the slope is sometimes reduced by lengthening the distance through which the fall extends by the construction of sills built for this purpose, or by reducing the amount of fall if the lowering of the surface of the pool above is not objectionable. In many cases, particularly in German practice, the more prominent irregularities of the natural profile are eliminated by reducing the greater slopes a very considerable amount by a liberal use of sills in connection with a progressive adaptation of the works of contraction to the conditions met; thus producing a more uniform velocity in the interest of navigation and sometimes securing a navigable depth which would have been impossible at the greater natural slope; such elaboration of the plan of improvement is, however, very much more expensive because of the greater extent of the works required and also for the reason that considerable additions or modifications are often needed to meet unforeseen effects due to so great changes in the river's regimen.

The location of the improved channel should, of course, be somewhere within the natural river bed in order to minimize the cost both of the works of regulation and of the excavation for deepening the channel when this is necessary; but where the width of shoal is to be reduced to perhaps one-third of its original spread and the variation in depth is slight and eccentric, just which part should be improved is a question for careful judgment. Economy of construction alone would dictate that the deepest portion of the shoal should form the rudiments of the new channel, allowing the contract-

ing works to be built in the shoalest water and thus requiring a minimum of material; this proposition may be correct, but is not necessarily definite and final.

Rivers of characteristically stable bed, at least to the extent that their shoals are rocky or of so solid a material that they mold the river instead of being formed by it, must have each contracted channel excavated through its shoal in such a way that it will not be too sinuous nor too sharply curved to be easily navigated, and its general alignment should be such that the current will keep it clear of sediment at low water and also at higher stages unless the velocity be great enough at low water to clear it of any temporary deposit. In rivers whose regimen is marked by a shoal-forming proclivity through insufficient velocity and concentration of flow at such places, not only are the needs just given equally essential, but because of the comparatively slight velocity at low water in this type of stream at such places, a more fundamental procedure is involved (of which the deepest water is an indication but not a proof); that of so locating the channel as to utilize such strength of the current as is necessary to effect the selfscouring of the channel where previously the stream had formed the shallows. This requires, not necessarily the place of maximum velocity of current at any section, but due regard to conditions above and below so as to secure the most direct and energy-conserving course for the stream especially at low water, but also at higher stages as well; for if a channel in this type of river is filled with sediment by currents crossing its direction at high water, conditions are not often such that there will be energy enough to effectively scour it out when needed at the low water stage. All these considerations are questions particularly requiring most thorough observation and study of each river in order to eliminate adverse conditions as far as is possible, and at the same time to take advantage of all favorable ones.

As an essential prerequisite is the determination of the natural course of the current at different stages, probably the greatest single aid in the necessary observations (one which is too often slighted because of the time and patience required) is that of submerged floats. These should be passed from deep water in the pool above to that in the pool below and their successive positions be located. A studied scrutiny of a full series of such observations will go far in determining the best average location for the improved channel. Undoubtedly many navigable passages which have trouble from filling with silt would have escaped much of that difficulty had

greater care been employed in their location. One example of this occurred about fifteen years ago on the Dnieper in connection with the contraction of a stretch of the river where the curvature was slight and the depth insufficient. The building of groynes outward from the opposite convex bank did not improve the channel at the concave side. Finally floats were used, starting several miles above and passing down-stream through the portion of the river in question. Their course disclosed the fact that the outline of the river above the ineffective works was such as to bring the strength of the current along the ends of the groynes instead of into the concave bank. Then additional works were built higher up-stream in order to throw the current into the shallow concave side, exactly as indicated by the floats; the result was then an entire success. An instance of an opportune "float" in connection with a different sort of channel planning is furnished by the fact that one pier of a great Mississippi River bridge was considerably changed from its contemplated position because a box was seen floating over the proposed site when the government engineer was inspecting the location!

44. The Relation of Axial Curvature to Channel Depth.—Thus due consideration of economy of construction, a course easily navigable, and a correct location of the navigable waterway are essential requirements in planning the regulation of rivers. But there is one more detail which profoundly affects the adequacy and excellence of the improvement; and that is the precise lateral outline given to the improved channel with regard to its curvature and the relation of each part to those immediately above and below. To make this important phase of control as effective as it may be, it is essential that the civil engineer should have a very intimate knowledge of the relation between every condition and its cause, and so to develop that clear apprehension of a river's actual nature and regimen which is so fundamental in planning its successful guidance and control. While law governs such relations between conditions and effects, its details so vary in different streams that each has its individual characteristics due to differences in regimen. Those effects which are the result of unusual circumstances or peculiar to especial cases are being recognized, and some of the general laws are becoming more clear. For many years engineers have felt the need of expressing empirically the general relation existing between the curvature of the banks, the velocity and volume of current, and other factors, and the stability and depth to be expected in channel development; but as yet with only partial success. The situation

still continues to be much as stated in a report to the Congress of Inland Navigation of 1894 which well summarized the proposition in the following terms:

"In the infinite variety of forms which she presents, nature shows us some in which the conditions which we seek to obtain are realized, and others wherein are found conditions which are harmful. The circumstances under which both kinds are produced must be studied and every effort must be made to reproduce those which are favorable and to set aside the others. Among these circumstances and in order to press closer together the questions referred to the section, the line of the banks will be particularly pointed out. It follows, from the observations which have been made and from the discussions to which they have given rise, that it is recognized almost unanimously that the line of the banks does exert a certain influence on the distribution of depths, but without its being possible to state this influence precisely by a mathematical relation between the curvature of the banks and the depth of the channel. And for a reason on which the whole of the section was agreed, this difficulty lies here: if the curvature of the bank is a very important factor in the distribution of depths, it is far from being the only one; it depends also upon the slope, on the width and resistance of the bed; it depends upon the character of the banks and the works; and, finally, it depends upon the concordance or the discordance between the high water and the low water channels. And the opinion of the section was that it was important to multiply observations on all these points. The section did not stop at this resolution or at the statement in principle of the undoubted influence which, all else being equal, the curvature does exert on the depth of the channel. It was of the opinion that it was highly important to act along the proper direction of the channel by the continuity of the latter in the crossovers from the curves of one bank to the reversed curves of the opposite bank. And it went still further. It seemed to the section that the principle of continuity, of which the suitability was first pointed out in connection with the curvatures mentioned in Mr. Fargue's studies on the Garonne, was further reaching and more generally applicable. Every sudden obstacle, every violent projection, every abrupt change in the direction of the water involves a loss of living force which is expended in a whirl, causes scours and produces disturbances in the regularity which so much effort is made to obtain. But this regularity cannot consist in a uniformity of which nature offers no examples; it can, however, be the result of continuity and it should be sought, not only in the shape of the plan, but also in the forms of the longitudinal profile of the cross-section as well as in the passage from the minor bed to the major; it is none the less necessary for the line of the works which should be so laid out that their action, little felt at first, should increase gradually to its maximum and then in like manner fade away."

However, as observation and experience increase, and especially

if civil engineers will publish the results of such studies more fully in regard to the details of conditions both when results are successful and when disappointing effects are noted, it will in time be possible to express the general laws derived in terms of the variables involved. Meanwhile, some of the general principles of design which experience now seems to usually corroborate will be given, but it must be understood that there are many exceptions to the modifications which may naturally be expected to result from definite channel forms, due to other contributing causes which the curvature alone cannot counteract; such as improper width, serious fluctuations in volume and velocity, or other instability of regimen, which cause shallows to form and difficulties to persist as a result of imperfect or incomplete control.

1. The curvature of the longitudinal axis should be sufficiently sharp to cause adequate depth of channel to form, and to hold the current from wandering from an axial direction. However, excessively sharp curvature tends to lessen the uniformity and stability of a channel, and makes navigation difficult. The adopted radius of curvature should be based on experience with other training works, especially those on the same or similar rivers, and on observation upon the natural stream where the low water channel depth is satisfactory. Theoretically it is believed that the radius of curvature should be directly proportioned to the square of the velocity in each bend, because of the resulting influence toward equalizing the velocities in successive parts of the river in the interest of its uniformity and stability; but the definite relation between these variables has not yet been empirically expressed. The general plan should involve a succession of curves of moderate radius, as uniform in general outline as is practicable. Examples of the curvature actually used in river improvement, under differing conditions, will be found in the next two chapters.

2. The type of axial curve adopted for the channel is frequently a circular arc, but there are advocates of a form which regularly increases in curvature to its vertex and thence similarly decreases to its end, as is the case with elliptical and parabolic arcs, lemniscates, double spirals and sinusoids, etc. There are some theoretical advantages in the use of the more complex curves, but whether the effects are materially superior is not yet definitely evident. So far as investigation has extended it seems that circular arcs are not as favorable in securing stability of conditions as are curves of gradually changing curvature; but there is no definite evidence of

any particular difference between them in the amount of deepening secured at shoal crossings.

3. The lengths of the curves of a river should be adjusted to its regimen, being neither too great nor too small; as indicated, again, by experience with previously constructed improvements and by observation of the river itself. One investigator has stated that this normal length is eight times the minimum width.

4. The width of a completely contracted channel should be adjusted to the curvature given to the channel axis, the width increasing somewhat as the radius of curvature decreases, such variations being effected gradually. Thus the maximum width will occur at the place of sharpest curvature, and the minimum will be coincident with the part most nearly straight. In the case of five well-regulated and permanently deepened crossings in the Garonne, the widths at the vertexes of the curves averaged nearly 40 percent greater than at the places of reversed curvature.

5. There should be no long, straight portion of channel; but if such straight portion be short and especially if it be between portions curving in opposite directions, a channel of nearly even depth may be developed in it; but such depth will be less than that in the curves above and below. However, continued experience on rivers of Holland indicated that even short straight portions should be reconstructed in curved form for the purpose of attaining the feasible increase of depth at these places.

6. There should be a studied continuity of design of the longitudinal plan, especially at shoal crossings in passing from the curve above to the reverse curve below. This fundamental principle is thoroughly expressed in the quotation last given, and its disregard marks the failure of many works of improvement, at least as first constructed. Shoaling is very liable to occur in poorly designed places of reversed curvature, as on the Waal, Hiwassee, and many other rivers on which this feature has been inadequately considered.

45. Some Early Experimental Investigations of Channel Form.— In no question involved in the improvement of rivers is there a greater occasion for requiring the assistance of experimental investigation to govern the design and to verify projected plans than in that of the curvature, width and correlated details of the proposed channel. The difficulties peculiar to such methods have generally led engineers to experiment by complete construction of the project; but any imperfection of design so detected is costly both in the expense involved in its modification, and in the embarrassment

to navigation while awaiting the attainment of the expected navigability.

Concluding from theoretical considerations of stream flow that the character of the action occurring at the bed of alluvial rivers is the same on a small stream as on a large one if the nature of the motion of the water is the same everywhere, Professor Osborne Reynolds made use of small scale models to verify the fact that conditions occurring in rivers can be essentially reproduced in experimental form. His first investigation of this kind was made in 1885 in connection with his study of the estuary of the Mersey, using a model whose horizontal scale was 2 in. to 1 mile and the vertical, 1 in. to 80 ft.; the form of the bed was molded in paraffine on which sand was spread to a depth of one-fourth of an inch. Over this bed the flow of water was induced in a way to be proportional in velocity and periodicity to that of the channel itself. The results so obtained were significant in reproducing to scale the essential characteristics of the natural waterway; and from this and other similar experiments, it was thought that a somewhat greater exaggeration of the vertical scale would be better. The investigator concludes that

> "This method of experimenting seems to afford a ready means of investigating and determining beforehand the effects on any estuary or harbor works; a means which, after what I have seen, I should feel it madness to neglect before entering upon any costly undertaking."[1]

This method of small scale experimental investigation to secure additional knowledge of the effects of proposed types of regulation works has been employed at different times by other European engineers, but much less frequently in fluvial channels than its possibilities would seem to warrant. One early instance of such a study was that of "La Petite Riviere Artificielle de Bordeaux," in 1875–6, which indicated some of the principles that have already been mentioned as important in guiding the design of works of regulation.[2] These experiments were made on nearly level ground at the side of a small stream from which the water for the investigation was taken at a point just above a dam, flowing through a 16-in. pipe fitted with a regulating valve into a head-basin about 7 ft. wide and 26 ft. long, which served the purpose of dissipating the

[1] Report of the British Association for the Advancement of Science, 1887, pp. 555–562.

[2] Annales des Ponts et Chaussées, I, 1894, pp. 426–462.

current velocities of entrance. From the head-basin the flow first passed through a rectangular channel section of masonry, 40 in. square; then through the experimental channel, from the lower end of which it flowed into a discharge-basin for receiving the sand entrained in its course. From this lower basin the discharge passed into the stream at a point about 250 ft. below the head works. The uniform slope of the course was fixed by the relation between the length of this artificial stream and the difference of elevation of the sills at its upper and lower ends. Although conditions were favorable for a direct measurement of the volume of flow, this was not done, but such volumes were later approximated by computation.

The sides of the experimental channel consisted of vertical planking. Its bottom was formed of sand from the Garonne about 1 ft. in depth, the surface of which was leveled before each experiment so that the effects of the flowing water could be determined everywhere. Various kinds of curves were used, and straight parts were introduced at places. The total length of the channel so studied varied from 195 to 213 ft.; its width was about 40 in.; its total fall was adjusted to 1, $2\frac{1}{2}$ and 3 in. at different times; and the radius of curvature of the circular curves was 32.8 ft., while those of the complex curves varied perhaps 50 percent from that value. Twenty-one different experiments were made altogether, and in them the surface velocities varied from about 0.7 ft. to 3.6 ft. per second; the minimum volume of flow was 2.22 cu. ft., and the maximum 9.54 cu. ft. per second; while the duration of the experiments ranged from 6 to 166 hours. Cross-sections were established at a distance of 3.28 ft. apart, and after each experiment careful measurements were made to establish the exact outline of the bed at each section and its elevations with reference to the original leveled bed.

Such were the details of the experimental channel made to verify effects that were expected to result from certain types of river improvement projects. The scale of reduction from the proposed works to those of the trial channel was about $\frac{1}{100}$ in horizontal and $\frac{1}{20}$ in vertical dimensions, $\frac{1}{2000}$ in area of cross-sections, $\frac{1}{3200}$ in volume of flow; the velocities were about 60 percent of those of the real river. The results gave positions of pools and bars exactly as anticipated, but the latter were shorter and at the crossings the current passed from one bank to the other more abruptly than had been forecasted. As for the realization of expected depths, those

observed after the tests in the pools are given as varying from 7.5 to 11.0 in., while that anticipated was 9.4 in.; at the shoal crossings, a depth of 3.35 in. was expected, but the observed depth averaged only about two-thirds this amount. However, some discrepancies in laying out the trial channel to fairly represent the natural water course, such as giving it too small a slope, etc., may account for some lack of proportionality between certain experimental results and those corresponding effects which were anticipated from observation and experience on natural rivers; while some of the differences presumably indicate probable tendencies and effects which it is the purpose of such experiments to disclose. Unfortunately, these observations were not definite and complete enough to furnish data to serve as a basis for a thorough and direct study of that especially elusive phenomenon, the actual movement of solid matter in erosion, transportation and deposition, in order to be able to express more definitely and precisely the principles and laws involved.

46. The Experiments on Models at Berlin.—The Experimenting Establishment for Hydraulic Engineering and Shipbuilding at Berlin has, in the last few years, made similar experiments on models to determine the best form to give the works of improvement in certain portions of the Weser, the results to be expected from a proposed plan of regulation at a place on the Vistula, and also to determine the desirable form of bank revetment on the Streckelberg near Usedom. The first-mentioned investigation was especially important, and a statement of methods employed and results obtained[1] is summarized in the following paragraphs.

Because of uncertainties with regard to the attainment of entire success in being able to definitely represent on a small-scale model all the conditions of a natural river and to be assured that the effects of artificial modifications in the stream are correctly indicated by corresponding changes of the model, it was decided that the first requirement was the true reproduction of a definite part of the natural Weser River, and then to compare the effects of flowing water upon it with the state of the river bed which had been produced by corresponding conditions. If this experimental verification of the correctness of the details of reduction in scale and choice of materials proved satisfactory, then it could be confidently assumed that experimental investigation of the effect of any proposed plan or detail of regulating works would show, on the model, the conse-

[1] Zeitschrift für Bauwesen, Jahrgang LVI (1906) Seiten 323–344, and LVII (1907), S. 67–78; Bl. 30 u 31, Jahrg. LVI; Wilhelm Ernst & Sohn, Berlin.

quences that would result from the same construction of full size in the river itself. For this purpose a portion of the Weser, 1.6 km. in length and shown in Fig. "*A*" of Plate I (facing p. 168), was

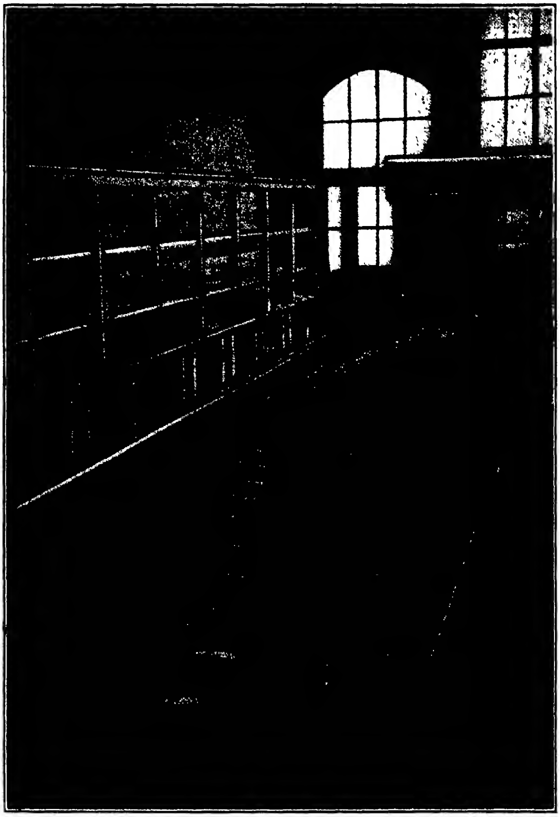


Fig. 22.—A part of the experimental channel.

chosen because of its availability in its definitely known characteristics both in its natural state and after works of improvement had been installed, the stability of its bed in showing similar conditions to exist at each recurring low water stage, and because a more effective regulation of that stretch of river was desired.

The condition of the facilities of the hydraulic laboratories caused

the adoption of a scale of linear reduction of 1 to 100 for both horizontal and vertical dimensions. With regard to the character of material which should be used for the bed of the artificial stream, it was evident that a variation in size corresponding to that of the river itself is important; but the question of the suitable proportionate size was not so clear. It was said that apparently the ratio of volumetric reduction should be as 1 to 100, which would call for an average diameter of grain of about 1.7 mm. inasmuch as that of the river gravel was about 8 mm. However, after some investigations of the behavior of graded sand of various average sizes under the action of flowing water, the effect of one of which is shown in Fig. 22, a river sand of an average diameter of about 1.2 mm. was chosen for the material of the model; this was, in later experiments, changed to 1 mm., or six-tenths the diameter which would keep the volumetric ratio of the particles the same as the linear ratio of reduction for the general dimensions of the model.

There remained the determination of the proportionate volume of flow and of surface slope to establish for the model. These two characteristics are interdependent. Four requirements had to be satisfied; a correspondence in relative height of mean low water, mean high water and mean water levels, the range in the model of course being one one-hundredth that observed in the Weser; all these water levels are to have a similar corresponding relation with respect to the tops of the groynes; the depths and cross-sections of the three water levels must have a like relation to those of the natural river; and the discharge in the model is to have a constant ratio to that of the Weser at all three stages mentioned. Recourse to both theory and trials was proposed for arriving at the definite relative values to be used. It was considered that the continued product of the slope, depth and the reciprocal of the diameter of grain should have the same value in both the model and the river.[1] The mean depth of the Weser at mean low water was 1.34 m., its surface slope was 0.00014 and the average diameter of the gravel of its bed, 8 mm.; the product of these three factors is 0.0000235. Similarly the size of sand grain of the model was 1.2 mm., the proportional mean depth, $\frac{1.34}{100}$; therefore the computed value of slope in the model would be 0.021. However, repeated experimental attempts to attain satisfactory results on the basis of that computed slope seem to have proved disappointing; at any rate a slope of about one-tenth that

[1] Zeitschrift für Bauwesen, 1900, S. 355.

value and a ratio of unit discharge of 1 : 40000 were experimentally found more satisfactory. Later, a surface slope of about 0.0015 and a corresponding discharge ratio of 1 : 50000 were found to produce a channel in the model still more nearly coincident with that of the river itself, especially for the higher stages.

Since the proportionate reduction in discharge was taken as 1:40000 and that of cross-section was 1:10000, the actual velocities of the model would be one-fourth those of the corresponding portions of the Weser. And since the linear distances are reduced to one one-hundredth, the rate of flow, with reference to the distances

FIG. 23.—Formation of bank of model.

traversed on the model itself, is twenty-five times as great as it should be. The scale of time selected was twenty-four hours to represent one year, purely as a result of trial, it being found that the leveled bed of sand was transformed by the running water into a miniature river channel in that length of time. Of course a continuation of flow, in the model, beyond the period stated would result in some further erosion and deposit of sediment, but its extent was so comparatively slight as to be considered unimportant. With reference to the reproduction of the changes in stage of river upon the model, the hydrograph of the Weser for the year 1897 was approximately followed, some of the minor fluctuations being averaged in the experiments.

Having thus epitomized the adoption of the scales of reduction, the methods of operation should be summarized. Small sacks of

shot, or small plates of slate, at slopes of 1 on 1, were used to represent those steep banks of the Weser which were protected, as shown in Fig. 23. It was also found necessary to define in the model the remainder of the banks which were not protected in the natural river, in order to prevent the model stream from running wild in all directions and widths; and for this purpose similar resistant materials were used at an inclination of 1 on 4. It was, however, soon discovered that the use of this invariable slope was defective, and for it was substituted one of two inclinations to more nearly correspond to the slopes of the river banks, the lower part being the steeper. It was found essential to extend these materials, used for defining the bank lines, well below the depth to which erosion would occur.

After outlining the banks of the model, the selected sand was spread between and leveled at the average elevation of the actual river bottom. Then the whole model was inclined to the desired slope, and the intended flow of water was introduced so that its effect in molding the channel could be observed. Such sand as was carried from the lower end was, after weighing, returned to the channel at the upper end in small quantities; but the method used in reintroducing it was considered objectionable, probably being one cause of some dissimilarities in the experimental results. Water levels were taken at every meter of length in order to accurately determine surface slope; and at the higher velocities three readings were taken at each meter point, one in the middle and one at each side, in order to eliminate lateral slope. Sometimes small surface floats and at other times Pitot tubes were used to measure velocities.

Cross-sections were taken so close together that a complete and precise outline of the river bottom was obtained; in fact this delineation was more exact than was that of the natural river, some irregularities which had escaped detection being afterward found as a result of their occurrence in the model.

Special experiments were made to determine whether the model works of contraction should be put in position before or after the formation of a channel by the running water; the trials indicated that the results were practically the same in either case. It was found essential to carefully extend such structures safely below the reach of erosive effects, especially at their channel ends. The materials used for these contracting works were small bags of shot and strips of tin, covered with a fine adhesive sand in order to roughen the surfaces; cement was first used, but was found to be less convenient and exact.

The verification of the correctness of the scale of reduction and other details of the model consisted in trial runs in which the water was caused to flow over the leveled bed in the manner and with the proportions already mentioned; and comparison could then be made between the form and depths of the resulting channel and those of the Weser itself. The first experiment of this kind was planned without the presence of the works of regulation. The form of bed produced in the model, with varying depths indicated, is shown in Fig. "*B*" of Plate I. Comparing this with that of the natural stream (Fig. "*A*") at once shows the similarity of experimental results in its governing features. Profiles and cross-sections also indicate a definite correspondence in general conditions. In the model the average of the depths is 1.41 cm. and the average area of cross-section is 105.2 sq. cm.; while the corresponding values of the river are 1.34 m. and 98.7 sq. m., respectively. However, there is found a lack of coincidence in details which suggests the conclusion that the system is not yet perfected. Such irregularities are found as differences in distribution of shallow and deep portions of the channel, the smaller depths in the reach shown by the model, the greater variability in the experimental depths, and especially the greater comparative depths in the concave banks. While the last-mentioned difference is not important with regard to the effect upon navigability, it is, nevertheless, one of the characteristics by which the question of the adequacy of experimental methods must be judged. The reasons suggested to account for these discrepancies of detail are partly the greater accuracy employed in delineating the bed of the model than in that used in the hydrographic survey of the river, and partly in the adopted choice of scale-reduction for size of sand, of velocities and volumes, etc.

A second test of the adequacy of the experimental method, as planned in detail as described, to reproduce on the small scale the conditions which the river shows, was secured by building into the bed of the model stream, as left by the experiment just described, fourteen groynes exactly similar in position, elevation and dimensions to those of the Weser itself, as illustrated in Fig. "*A*" of Plate II (facing p. 170). The result of this experiment is shown by comparing the conditions produced in the natural river with the results exhibited by the bed of the model in Fig. "*B*" of Plate II. In this case there is again a good correspondence in general characteristics but accompanied by exaggeration of certain details in the model which the excessive relative velocities, or other unperfected assumptions

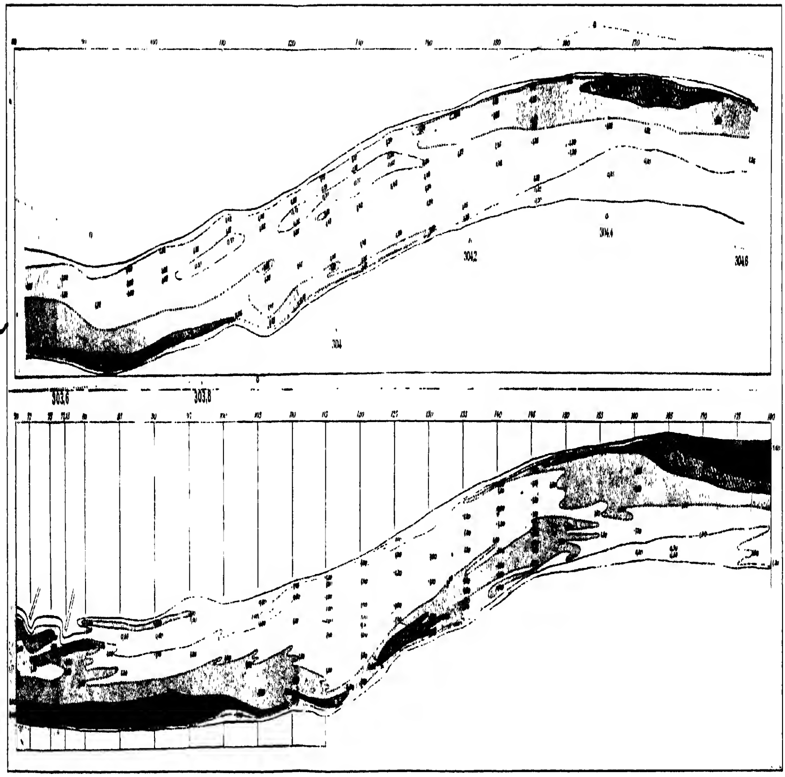


Fig. "A."

Fig. "B."

(Between pages 168–169)

for the small scale reproduction of conditions, are probably responsible for. Both the river bed and that of the model show the improvement of the channel at the shoal crossing produced by the contracting works. In fact, the worst place in the river channel, that at the first groyne on the right bank where a deep but narrow and crooked pocket occurs, appears also in the model in an intensified form. In both figures the shoal part extends from the left bank well toward this place, and the resulting eccentricity of the channel at groyne 1 is largely attributable to the irregularity of the shore line here. Taking into consideration all the facts of the tests for the verification of the adequacy of the experimental procedure, it was considered that the results obtained by the service of the model were, in general, deserving of credence.

Other evidences of a corroborative character are the facts that careful measurements of the lateral slopes in the sharp curves of some of the model experiments showed a satisfactory agreement with theoretical values; that computed values for these small streams of the "*c*" of Chezy's formula ranged from about 38 to 48 (with the metric units used) for the different volumes of flow; and that the substitution of other material for the erodible bed has been found similarly satisfactory in reproducing the essential characteristics of river channels naturally formed in corresponding soils.

Having thus proved the sufficiency of the small scale operations to adequately indicate the general effects which similar conditions had produced in the river itself, numerous experiments were made to show what situation, dimensions, form and other characteristics of contracting works would be most effective in the further regulation of the Weser. The most significant series was that in which methods were investigated to secure an increased depth of channel through the shoal crossing.

Experiments were made to disclose the effect of height of the groynes above the low water surface upon channel depths, using corresponding variations in volume of flow in each case. Two different heights of crest were used; in one it was fixed at 0.95 cm. above low water, which corresponds to about 3 ft. in the natural river, and in the other, about half that amount. The difference in results for this moderate variation was slight. There seemed to be a tendency toward an increase of the mean and the maximum channel depths, and also of those of the controlling section at the upper den, by the use of the lower groynes when they were short; but when they considerably reduced the width of the stream, the

higher ones seemed more effective. The only exception to this general tendency appeared to be the greater maximum channel depths occurring also with the lower structures in the case when they were long. The deepening was greatest at the lower end, the change at the upper being relatively small. Therefore the higher structures manifested a general influence toward the increase of irregularity of the channel profile, which is without benefit to navigation unless the minimum section has also been deepened. In general, the lower crests gave a smoother river bottom, with a reduced development of eddies and eroded pockets.

Trials of various slopes of the channel ends of the groynes were made, at inclinations varying from 1 on 4 to 1 on 10, the other characteristics of course being kept constant. The general result was an increase of local scour and of channel depth at the lower end of the stretch as the ends were made more nearly vertical, with slight if any increase at the upper end. Hence there was shown no advantage of the steeper end slopes in improving the navigable depth, while the flatter ones favored the development of a more regular channel bed and the minimizing of objectionable velocities and erosive action in the vicinity of their ends.

The fourteen groynes thus far considered were located in the two successive concave banks of the stream, and in only two instances were they opposite. In Figure "*C*" of Plate II is shown the effect upon the channel of introducing additional groynes to complete the contraction from both sides, so that each of the original structures now had its complementary one at the other bank; and there was also placed an additional groyne (1a) at the right bank, up-stream from the previous series so as to lessen the abruptness of the entrance to the regulated part and so to reduce the eccentricities of current, the crookedness of channel and the depth of the pocket which previously marked the vicinity of groyne number 1. The channel width was made 60 cm. and the height of groynes above the low water surface was fixed at 0.45 cm., still adhering to the scale of 1:100; the slope of their ends was 1 on 4.

The effects on the channel of reducing its width 8 percent are shown in Fig. "*A*" of Plate III (facing p.172), and those of a reduction of 16 percent appear in Fig. "*B*" of Plate III. As would be expected the depths are, in general, increased by this contraction; but some of the details of distribution of such increase are important. In the vicinity of the upper end of the narrowed channel, the place which more than any other manifests a persistent recalcitrance in

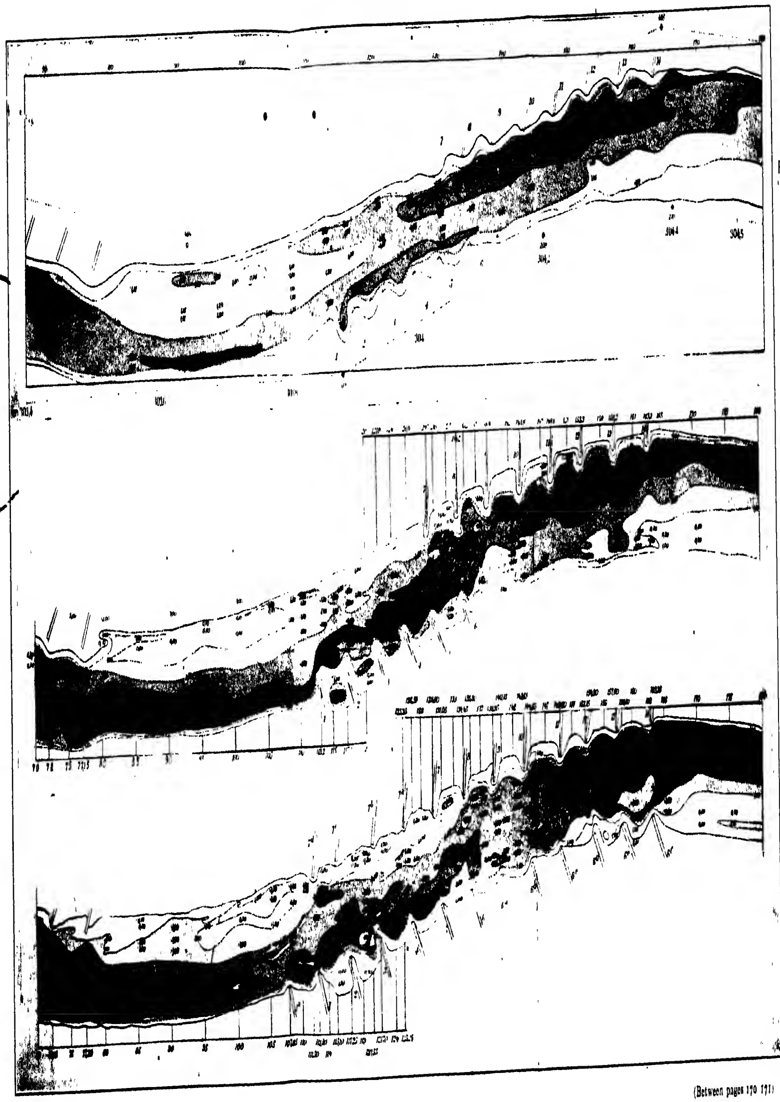


FIG. "A."

FIG. "B."

FIG. "C."

(Between pages 170 171)

river improvement works of this kind, there resulted a slight decrease of depth for the conditions just given; but with groynes extending twice as high above low water, or with those having a slope of 1 on 8, this depth was somewhat increased by contraction in both cases. In every instance there was a noticeable deepening in the middle portion of the regulated stretch, and a considerable increase always occurred at the lower end, where the slope of the water surface becomes largely concentrated by the modified conditions. It is also true that narrowing the channel did, in every case, greatly decrease the depths in the concave bend above, and the pockets near the ends of groynes were usually deepened and enlarged. These latter facts would not affect the navigability of a river, but they illustrate the extent and the variety of the influences which accompany, and even sometimes supersede, the direct effect sought.

Fig. "*C*" of Plate III (facing p. 172) shows the results of experiments to develop the effect of a gradual reduction in channel width from the ends to the central portion of the regulated stretch. In this case the middle part was 55 cm. wide, the entrance was 13 percent and the lower end 9 percent wider, other details remaining as before. This arrangement afforded as great a depth of channel in the middle part as in the lower portion of the improvement, largely eliminated the pockets, and produced a much more regular channel form. The conclusion is reached that a gradual transition from the wider, unregulated part of the stream toward the place of maximum contraction, followed by a widening toward the down-stream termination of the works, is desirable to secure a greater permanency of regimen and uniformity of current and of the river bed, as well as to minimize the tendency of the contracting works to deteriorate the channel above them, even to an extent which may make it shallower than the improved part and so require an extension of the improvement up-stream. This influence of contraction upon channel conditions was found to noticeably extend above and below the works for a distance averaging about eight times the contracted channel width, at the existing slopes and other conditions of the model.

The details of these experiments have been so fully summarized in order to illustrate the complex and intricate character of the effects produced by even small variations in works of river control, to impress the fact that a relatively small modification in plans often produces material changes in the stream bed, and to emphasize the opportunity offered to thus test in advance the probable effects of

proposed improvements in detail so as to avoid unnecessary expenditure resulting from either faulty design that would necessitate reconstruction or inadequate plans that would prove incapable of accomplishing the desired improvement. Especially should the statement of such experiences give opportunity for others to profit from their successes and to improve upon details of doubtful efficacy, until the procedure may become standardized. It has been proposed,[1] for example, to use scales of reduction which would result in a greater resemblance of conditions of the model to that of the stream than any thus far used, and which would at the same time preserve theoretical relations undisturbed. The suggestion involves the adoption of the same surface slope, in the experiments on the model, which the natural river has at the place represented. Then if the linear scale of reduction were 1:100, the volume of flow should be 1:100,000, and the relative diameter of particles composing the bed and banks should also be as 1:100. The resulting velocity would then be an actual one of one-tenth that of the natural river; or, relative to the progress through the experimental channel, its velocity would be ten times that of the river itself. There would seem to be a probable superiority of these last figures, compared to the respective velocity values of four and of twenty-five of the Berlin model, in securing a more reliable correspondence in effects produced in detail. The only apparent difficulty of these proposed scales of reduction concerns the size of the particles of earth or sand, which become very minute; those for the Berlin experiments would have had to average 0.08 mm. in diameter, or only slightly larger than the apertures of a standard 200-mesh cement sieve. The difficulty and expense of procuring a sufficient quantity of so fine a material is evident, especially when the importance of a corresponding gradation of sizes is remembered. Perhaps this will cause a return to a less reduction in the vertical than in the horizontal scale for future model experiments, making the former two or more times that of the latter. It will be recalled that the Berlin experiments did, in effect, use a larger scale for representing vertical dimensions in fixing the surface slope to employ, but not for the other vertical values such as depths.

47. Advantages of Trial Works in Rivers.—Engineers have more often, in cases of apparent uncertainty, made experimental models of full size; that is, they have at times constructed contracting works at a single portion of the river in such a way that they could be readily modified; and have noted the effects, changing the plans as found

[1] Proceedings, Institution of Civil Engineers, Vol. 146, pp. 216-222.

PLATE III.

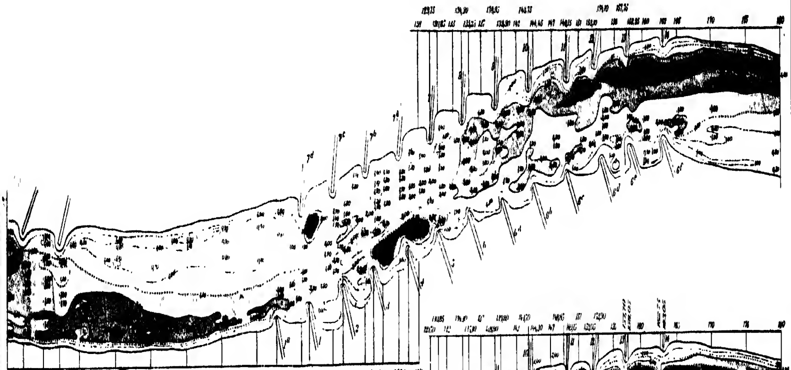


FIG. "A."

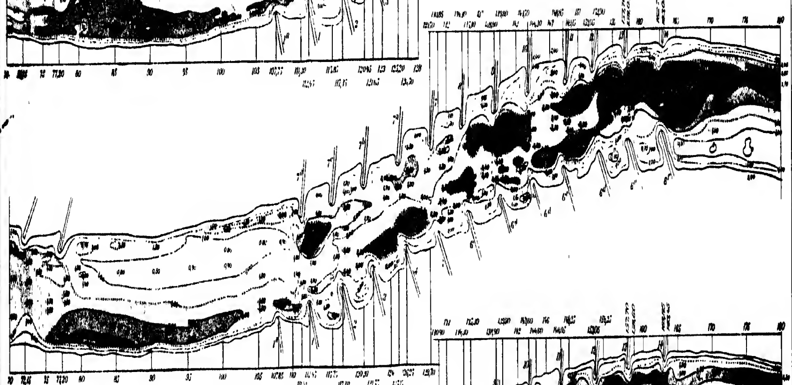


FIG. "B."

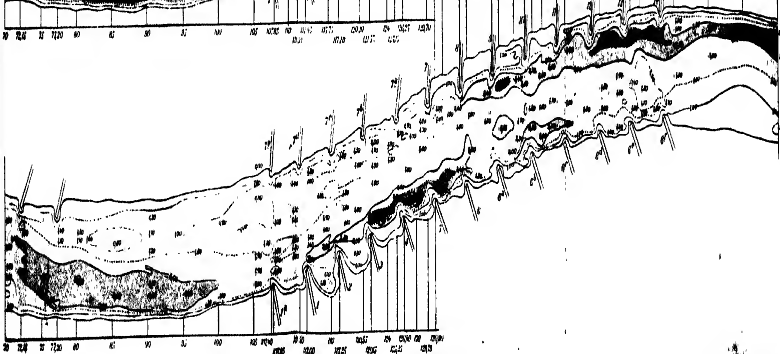


FIG. "C."

(Between pages 172-173)

necessary, before finally deciding upon the complete design. This procedure was followed, for instance, in carrying out the improved regulation of the Oder River. The necessity of utilizing, in the most effective way, the capabilities of the limited low water flow led, in 1905, to the securing of an appropriation of $250,000 for the construction of works on three trial stretches about 6 miles in length, each, as preliminary to the complete regulation of the entire distance of 186 miles. These three experimental portions were finished in 1911, and indicated desirable details in perfecting the plans of regulation for the entire distance. While this method is more expensive and requires more time than does recourse to model experiments, it is more sure to give precise results. Paper 10 of the Twelfth International Congress of Navigation recommends:

"When there are not river reaches which can serve as an example and owing to their nature have remained stationary for several years, it is then to be recommended to reduce the widths gradually on a trial reach in order to arrive at a proper distribution of width and depth on the cross-section necessary for the flow."

Such a procedure is far more economical than is an extensive changing of constructed works, as has sometimes been found necessary, and brings a surer success.

Among the conclusions adopted by the delegates to the last mentioned Navigation Congress (1912) are the following, with reference to the necessity for further studies and the methods recommended for their prosecution:

"That hydrotechnic laboratories intended for the study, on small scale models, of the life of rivers become of more and more extended use and that they be supplied with the means necessary to experiment with the various processes for improving the navigability of rivers and, in so far as possible, in connection with the studies and works carried out on the rivers themselves.

"That the resolution of the Sixth Congress of Inland Navigation, voted at The Hague in 1894, be carried into effect, this resolution calling for taking up, in connection with rivers having but one current, the study of a short, clear formulary, which shall be sufficiently complete and include the information necessary to define the characteristics of every river studied, from the double point of view of its regimen and its navigation.

"That the improvement of the navigability of rivers having but one current, completed by those of the laboratory experiments and of the formulary, be kept on the order of business of the next Congress of Navigation."

CHAPTER VI

WORKS OF CHANNEL CONTRACTION

48. Conditions Affecting the Location of the Channel.—The channel of a river is especially liable to be deficient in depth at places where islands divide the stream into two or more arms. A comparatively inexpensive contraction and resulting deepening is obtainable by closing all but the selected channel to the low water flow of the stream. This is effected by the construction of a traverse (sometimes called a dike, cut-off dike, dam, cross dam, closure dam, etc.) across the arm whose volume is needed to swell that of the chosen route; and its location is often across the middle portion, where the width and depth are small and the structure is more protected from the current than it would be at the upper end. The tops of traverses are usually low so as to minimize their obstruction to high water flow, and their construction, materials, costs, etc., are similar to those of groynes and training walls as described later in this chapter.

The increase of the low water depth secured by concentrating the flow into one channel is primarily produced by the rise of the river surface in consequence of the increased volume it now accommodates. However, this immediate effect will not be permanent if the resulting velocities are eroding ones; but the bed will then be lowered, and the surface will follow, until virtual equilibrium will again be established. The elevation of the water surface may also be prevented by excavating the channel bed if the strength of the current is insufficient to produce this result. In any case the effect of closing one or more arms is to increase the depth of the one remaining open.

The question as to which shall be selected as the navigable channel is one requiring a thorough consideration. If the velocities are undesirably high in the shorter course, they may be minimized by choosing the longer. In case the deficiency in depth is slight, the closing of the narrower "chute" may be sufficient; but if a considerable deepening is needed, the low water flow of the wider branch may be advantageously thrown into the narrower one, and so reduce the

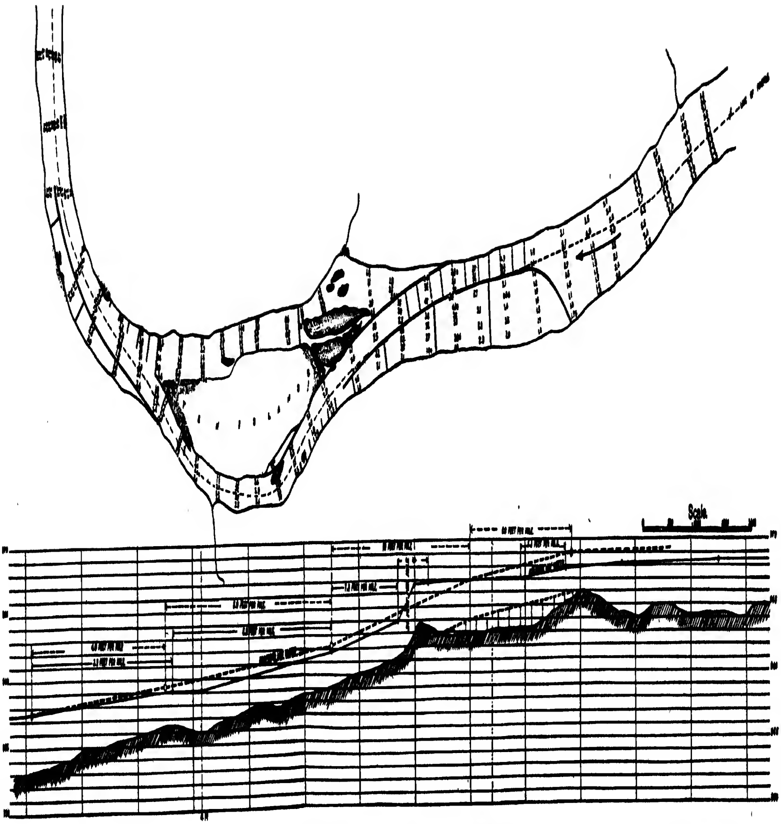


FIG. 24.—The improvement of Ten Islands' Shoals.

(Facing page 174.)

extent of contracting works that might otherwise be needed, or even entirely avoid the necessity of their construction. However, caution is needed to avoid the difficulties which sometimes follow an excessive narrowing at such places. It is stated that the contraction of the Garonne was carried so far, on portions of that river, that excessive erosion was produced, resulting in finally depressing the low water surface as much as 4 or 5 ft. and thus developing new shoals above; this effect could have been avoided by planning a less radical contraction, choosing for the channel those arms which were the longer.

Another essential consideration is the selection for improvement of that channel whose position, curvature and course are such as to both offer the best facilities for its easy navigation and to furnish that sequence of directions and curvings which are more effective in securing and maintaining the required depths and widths. The latter proposition involves a thorough apprehension of all those principles of control which are discussed in the chapter immediately preceding.

An example of the plan of improvement of a river, in the vicinity of islands which divide its flow, is offered by Ten Islands' Shoals of the French Broad River[1] as shown in Fig. 24. The traverse is seen extending from "Ten Islands" to the right bank. Training walls and groynes are employed to define the channel both above and below the selected channel to the left of the islands; and sills are indicated in the upper portion of this improvement, the purpose of which is to raise the water surface, and so increase the navigable depth, about a foot. The general regularity in outline and systematic succession of curvatures is apparent. Perhaps trouble may occur just below the foot of "Ten Islands" where a very slight curve to the left lies between two sharp curves in the same direction, and also at the very head of the improvement where the training wall on the left of the channel joins the left bank rather abruptly; but the doubtful effects of these details are much less likely to be serious in the case of this river with its rocky bottom than would be true if it had an unstable bed.

Whether the improvement is effected by closing superfluous arms at islands or in the more general way of building contracting works, a necessary preliminary is the determination of the desirable width of channel, and for this purpose the aid of the formula $b\sqrt{d^3} = \frac{Q}{c\sqrt{s}}$ is very useful. However, there exists an indefiniteness in its appli-

[1] H. R. Document No. 616, 56th Congress, 1st Session.

cation which is partly mathematical and partly practical. The former is mainly caused by the uncertain value of the coefficient "c," which ordinarily varies from about 50 for the shallower channels to 70 or more for deeper ones; and the latter results from the uncertainties in estimating the relation between the mean depth of the formula and the navigable depth which will actually be secured, the loss of volume through leakage of the contracting works, etc. So considerable is the total difference between the theoretical value of the second member of the equation and the actual channel dimensions which can consequently be given to the terms of the first member, that it is always necessary to provide an ample proportionate surplus to escape disappointing results. How much this excess should be is a question which must be decided for each case considered. Unfortunately the published statements of the complete hydraulic elements involved are comparatively meager, especially with regard to those of improved channels. This condition renders it impossible at the present time to tabulate results in a way to be of definite value; but the situation may be summarized by the statement that accessible data indicate that a figure for "c" between 30 and 80 percent in exceess of its theoretical value is frequent. This may be otherwise expressed by stating that, if the navigable depth is represented by the "d" of the formula, its attainment in the river seems to require the use of a value of "c" which averages about 50 percent in excess of its actual magnitude in computing the amount of contraction required.

In addition to the degree of excellence with which the improvement fulfills its primary function of confining the low water flow advantageously to the improved channel, those regulating works are the best which most completely exemplify economy of cost, permanence, safety to boats navigating the river, adaptability to modification as expanding navigation requirements or other conditions make necessary, prevention of loss of needed volume of flow through the contracting works, reasonable regularity in slope and velocity in the improved channel to facilitate its navigability and to safeguard the works against undermining, and encouragement to the deposition of silt and detritus in the obstructed portion so that an artificial bank may form to more effectively confine the river to its permanent navigable channel as planned, especially after the structures have become weakened in service.

49. The General Planning of Works of Lateral Contraction.— Contracting works as usually built consist of two distinct types,

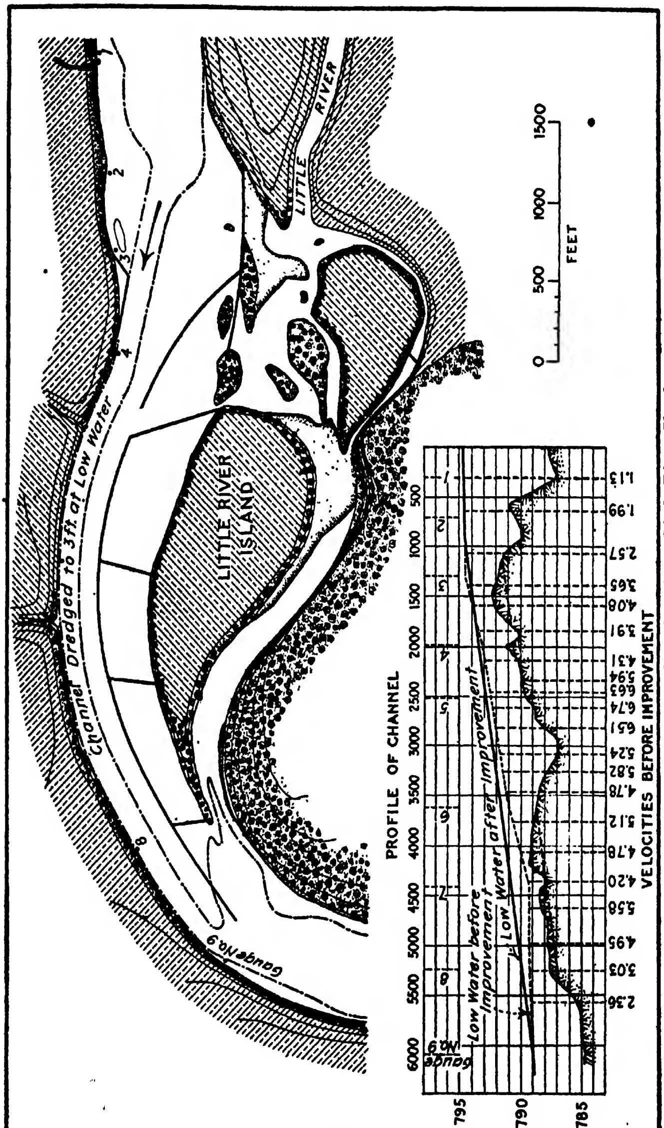


FIG. 25.—Plan of works at Little River Shoals.

groynes and training walls, each of which has its advantages as largely determined by the conditions to be met in making each improvement; generally a combination of the two kinds is found advisable, as on the Danube, Rhone, and Mississippi Rivers. They are built at both margins of the proposed channel when the improvement of a shoal crossing is undertaken or when the river bank, along which that channel may be advantageously formed, is too irregular to be used as one of its sides. Revetting to prevent erosion of the exposed bank is, however, often necessary when the contraction is made from one side only. The advantages of narrowng mainly from

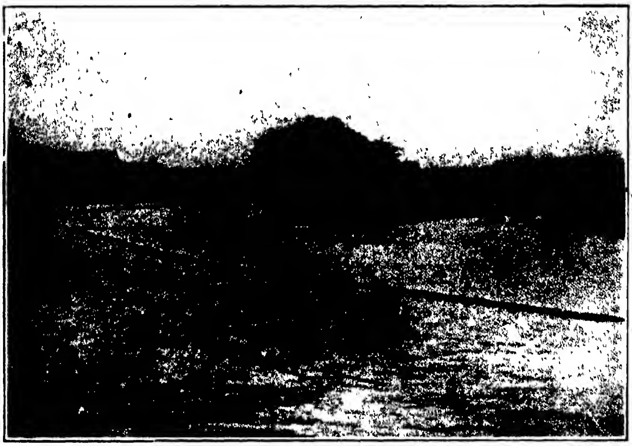


FIG. 26.—View of Little River Shoals.

one bank are a usually greater ease and safety of navigation; and a greater economy in first cost and maintenance, as well as a superior channel, results when the deeper concave side is chosen for the waterway. An example of this usual practice is shown in Fig. 25[1] (p. 177) in which the groynes and training walls of Little River Shoals of the Tennessee River were constructed from the rock blasted and excavated from the channel, and the low water surface slope was so much lessened that steamboats now stem the current without especial trouble; the attainment of the project seems permanent, even without contracting works at the upper end. The appearance of the finished improvement is shown in Fig. 26.

[1] From Brochure 4 of the International Congress ot Navigation, 1912.

The conflicting interests of economy of construction and effectiveness of current action constitute a condition which requires most careful consideration. There is no doubt that a superior channel is obtainable by a scrupulous adherence to the principles involving a very gradual change in width in approaching a contracted section, and in planning the longitudinal outline as a continuous succession of regular curves exhibiting a reasonable similarity in length and in the rate of curvature, suited to the regimen of the stream. The practical necessity of adjusting the plan to the natural river bed limits the freedom with which these principles may be employed; and the question of expense, which increases greatly as the scope of a project is extended toward complete realization, is necessarily prominent. Yet one may greatly increase the effectiveness of the improvement by adhering to these principles as definitely as conditions will allow,

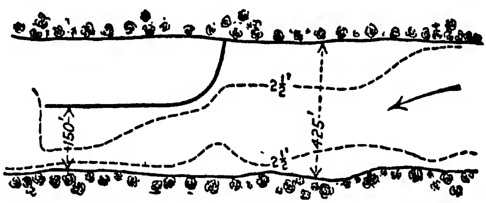


FIG. 27.—Channel outline at Bunker Hill Shoals.

often without increasing the cost of a project materially above that which would be involved in a defective design. The more stable the bed of a river may be, the less amply and thoroughly will its channel form respond to the character of its control; and yet current action is so persistent that the outlines of the bed will show its molding tendencies to be in thorough accord with the laws of stream flow, even in piedmont rivers. Fig. 27[1] is a sketch of the improvement at Bunker Hill Shoals of the Hiwassee River which is here suddenly narrowed to 150 ft., by a training wall connecting with the right bank, in the endeavor to obtain a navigable depth of 30 in. at the low water stage. There is not enough of the river shown above the shoals to permit the molding influence of the current upon the upper part of the navigable channel to be traced; but the characteristic effect of the abrupt contraction, deflecting the currents sharply toward the left bank, is plainly seen in the gravel shoal still extending from the lower end of the training wall nearly to the left bank. In

[1] "Professional Memoirs," Vol. 4, p. 213.

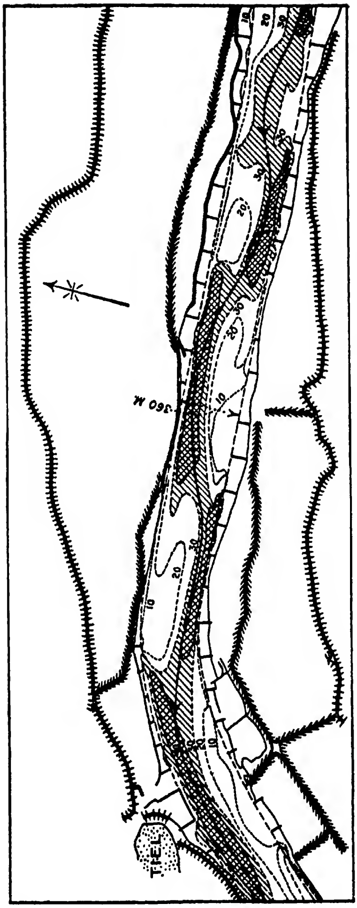


FIG. 28.—A portion of the River Waal, in 1889.

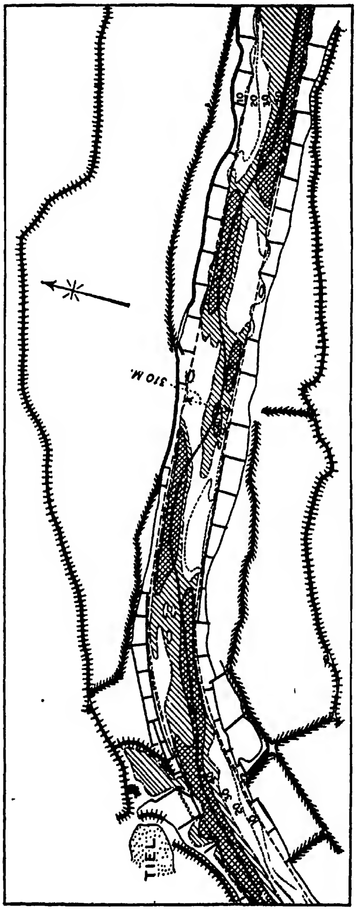


FIG. 29.—A portion of the River Waal, in 1896.

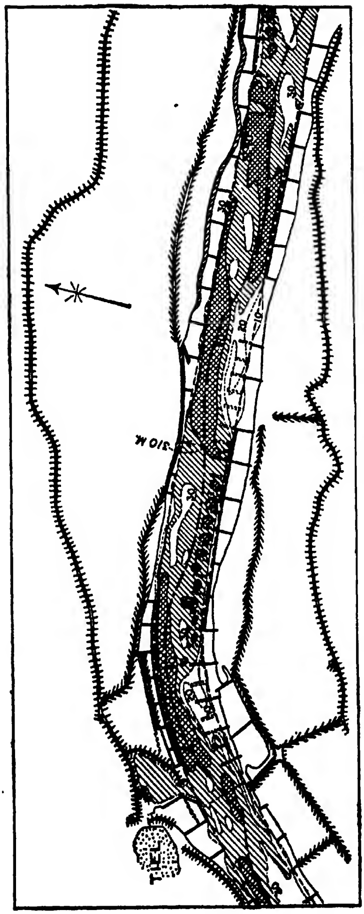


Fig. 30.—A portion of the River Waal, in 1908.

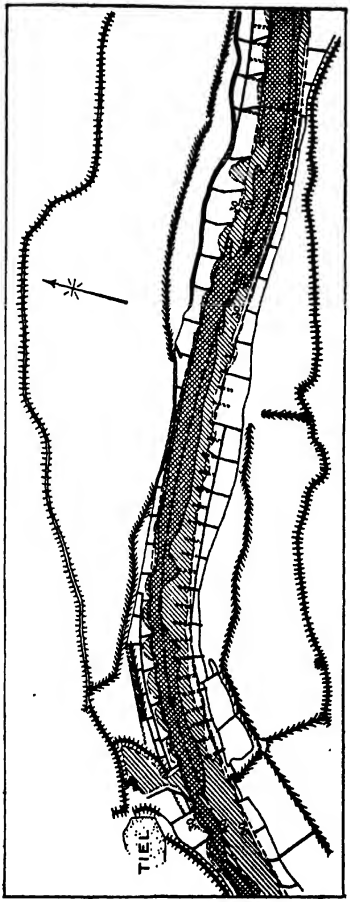


FIG. 31.—A portion of the River Waal, in 1911.

other respects the improvement has been successful, reducing the maximum slope from more than 13 to 9 ft. per mile, and the maximum low water velocity to less than 4 ft. per second.

The reconstruction of works of lateral contraction to secure a better channel by correcting defective outlines has sometimes been found necessary. One experience of this kind occurred in the River Waal near Tiel.[1] The bed is composed of a sandy alluvium, and is therefore very responsive to the molding influence of the currents as affected by the form of the works of regulation. Fig. 28 (p. 180) illustrates the condition of this portion of the river in 1889, after the bed had been narrowed by groynes at both sides to a uniform width of 1181 ft.; the maximum continuous depth obtained is seen to be barely 10 ft., and this is only attained in a tortuous channel which includes five crossings in a distance of about 2½ miles. It was then proposed to improve the navigability by contracting the width to 1017 ft. by extending the groynes, and by connecting their ends by training walls in apparently important parts of the stretch. The result as shown in Fig. 29 (p. 181) was a general deepening of the channel, it is true; but not only did the modification in width fail to reduce the number of crossings, or to end the abnormal condition of deep water occurring at the convex bank, but it actually resulted in lessening the navigable depth. This expensive reconstruction having failed to produce the desired effects, sills were built along the left side as shown by the broken outlines in Fig. 30 (p. 182). Their crests were 10 ft. below the low water surface and their object was to throw the current toward the right bank in order to straighten and deepen the channel; but "the results obtained were altogether insignificant in proportion to the expenditure." Its tortuous character was not improved, the navigable depth was not increased, nor was the winding course even yet fixed in position as the mobile bed continued its erratic behavior which resulted in a sill being at one time buried in sand and a few years later having perhaps 15 ft. of water each side of it as the shifting channel endeavored to displace it. During all this time the required depth was maintained by dredging. Finally, it was determined to reduce the width to 853 ft., and at the same time to replace the straight stretch of river in which these difficulties occurred by a continuously curving form. This transformation was most simply accomplished by contracting at the same time, because the groynes could thus be extended to conform to the new curved outline decided upon. The excellent effect is in-

[1] Brochure 10 of the International Congress of Navigation, 1912.

dicated in Fig. 31 (p. 183), which shows a regular channel nots les than 13 ft. in depth lying at the concave banks, with only one crossing and that a very gradual and easily navigated course. Inasmuch as the final narrowing occurred at the same time that the change was made from a straight to a curved outline, it is impossible to exactly distinguish the effect attributable to each factor. Yet because the previous contraction did not reduce the number of crossings of the channel in its straight reach and also because of experiences elsewhere repeatedly proving the salutary influence of a correctly curving longitudinal form in holding the channel continuously in its concave bank, it may be safely concluded that the general effect of the modified outline was to eliminate its tortuous character and fix its position, while the reduction in width aided in increasing the depth.

With regard to the changes of curvature it may be noted that the straight reach was replaced by a curve whose radius is about 2½ miles; and the curve at Tiel, whose original radius was about 1½ miles, was modified in the project of 1909 to a radius of about 2 miles. Even this curvature may be somewhat too sharp, as the channel at Tiel appears too narrow in the last figure. The experiences here outlined must not be adopted too rigidly. For instance, although sills did not accomplish any material benefit in the circumstances existing in this particular improvement, they have been efficacious in other cases. The type and the details of the constructed works were, in the end, successfully adjusted to the regimen of this river. The commanding influence of a correct planning of bank form seems evident.

To remember the curvature which proved successful in the case just described; or to know that the radius on the Elbe is usually about ½ mile, though sometimes it is less than ¼ mile; that its value for the more effective curves on the Garonne averaged about a mile; or to determine the typically advantageous curvature for other rivers without at the same time obtaining all their characteristics which simultaneously contribute to the attainment of satisfactory channel conditions, is likely to prove confusing and even misleading if the attempt is made arbitrarily to profit from these experiences in the planning of a project for another river. It is only when the relation between an effective curvature and all the hydraulic elements of the stream at the same place is disclosed, that such information becomes definitely available. Unfortunately such complete information is hardly obtainable in any instance. Even the more important values of velocity and volume rarely appear in connection with those of the radius of curvature which has been found effective in holding the

currents under such control that the channel is neither deficient in depth and regularity of position because of insufficient curvature, nor too restricted in breadth nor too erodible on account of excessive curvature; and either extreme increases the difficulty of adequately improving the shoal crossing at the place of reversed curvature below. Hence the opinion, repeated by the International Navigation Congress of 1912, that care should be taken by engineers interested to systematically formulate the characteristics of rivers for the purpose of making possible a thorough study of the relation between the curvature of the banks, the depth of channel, and the other elements of regimen involved.

50. Comparative Serviceability of Groynes and Training Walls.— Training walls[1] (often called longitudinal dikes, training dikes, longitudinal dams, jetties, etc.) have a direction parallel to that planned for the current, and typically extend throughout the length of the contracted section. Groynes[1] (also named contracting dikes, transverse dikes, cross dikes, spur dikes, spur dams, cross dams, wing dams, spurs, jetties, weirs, etc.) on the contrary are typically built from the bank outward to the contracted section in a direction approximately perpendicular to that of the regulated current, their outermost ends thus terminating at the margins of the proposed channel. The number of them at each shoal depends mainly upon the length of shoal necessary to improve, although its width, the velocity of current, etc., also have their influence. In the case of a training wall there is a complete continuity of influence paralleling the improved channel at the side, which groynes alone unavoidably fail to secure; and therefore they cause a greater or less concentration of the slope, and a consequent increase of velocity, opposite and just below their ends, the amount of such variations through the length of the shoal depending on the distance apart of the groynes, other conditions being constant. In regard to uniformity of velocity and continuity of influence on the current, then, the training walls are decidedly superior; and this is an important attribute, being the particular one in which they especially excel.

On the other hand, the loss from leakage is generally less in the typical case of groynes because their spacing must be so reasonably close in order to prevent excessive concentration of slope and velocity, that the difference in surface elevation of water on the two sides of

[1] For the sake of clearness, a single name has been always used to designate each type of construction herein discussed; and the word selected in each case has been chosen because of its distinctive character.

any such structure is comparatively small; while in the case of a training wall, if it only connects with the bank at its upper end in order to collect the volume of flow into the channel, the surface of the quiet pool behind is considerably below that of the channel except at the lower end, and the loss by leakage at low water is correspondingly great, increasing with the length of shoal. This loss may be a serious matter in rivers of small minimum discharge.

At higher stages of a stream, when a considerable volume of water would be flowing over the top of solid contracting works, there would result such a current in the space behind a training wall

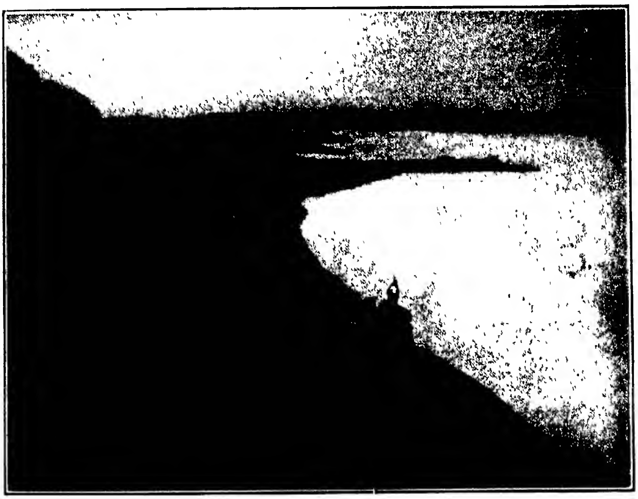


FIG. 32.—Accretions induced by groynes.

that the filling of it with depositing sediment would be very unlikely; while with groynes, built as they are transversely to the direction of such flow, the conditions would encourage the accumulation of silt outside of the improved channel. The experiments of 1907 with the Berlin models and practical experience both indicate the general fact that the fill between groynes in the concave side, when they are used without a training wall connecting their channel ends, occurs in the typical form of tongues extending from shore, each one enclosing a groyne; while the space between groynes remains largely open due to the eddying currents produced, especially when the

groynes are short; on the convex side, however, the sedimentary deposit is very much more complete, as indicated in Fig. 32 (p. 187.[1]) More than a half-century ago it was noted that the accretions, between the groynes built on the middle portion of the Garonne, were extensive and quickly secured; the spaces filling to the level of their tops in five or six years for a total area of about eight-tenths of the area occupied by the groynes, or more than fifty acres of reclaimed land per mile of contracted channel on this small river.

The foregoing considerations in particular have led engineers to very frequently combine groynes with training walls, the required number of the former being less than when no training wall is built connecting their ends. This combination unites the principal advantages of both types. When this arrangement is adopted, the name cross dam, tie dam, check dam, traverse, etc., is sometimes applied to the groynes.

Training walls are considered less safe to navigation at medium stages of the river because of the less distinct indications of their position to the pilot and the greater strength of current tending to draw boats upon them. Groynes are easily shortened if experience shows that a channel has been too much narrowed, or lengthened if further contraction is found advisable. Yet this question of possible subsequent adjustability is not so definitely adverse to the training wall type as it might seem to be, if only the original design has placed it so that the channel is amply wide; in this case it is perfectly practicable to extend from it short spur groynes to secure a further contraction; but if the training wall were later found to contract the channel too much, there is no remedy but its entire removal and a complete reconstruction. The method of contracting a channel conservatively so as to avoid the difficulties which follow an excessive narrowing, is quite common in experience. One instance is that of the Waal, just referred to, in which the contracting works were extended twice before the desired results were obtained. Another is that of the Elbe on which the channel width was tentatively fixed but later was several times reduced, and even now a further narrowing is under consideration.

Groynes and training walls being both generally constructed of the same material in any locality, there is practically no difference in their durability. As far as their permanence is affected by the relative likelihood of destruction from scouring velocities, there is no particular advantage of one type over the other; because careful

[1] Report, Chief of Engineers, U. S. A., 1895, p. 4049.

design, with especial regard to a properly spread foundation mattress, will render either kind stable against the conditions to be met. But an advantage exists in regard to the opportunity offered for the contracting works to become embedded in silt and detritus, which is of more or less importance, depending upon what material is used in the construction and the details of the design; whatever preference exists, on this score, favors the groynes.

As for cost, a narrow shoal favors groynes, and a wide one, the training wall construction; and yet even this generalization is not at all absolute because of various other conditions to be met, as above discussed, which generally encourage a combination of the two types, especially adapted to local conditions. In general either kind of construction is applicable to streams of moderate local slope; as groynes characteristically used on the German Elbe, Oder and Vistula, and training walls applied to the improvement of the Saxon Elbe and the German Rhine. Where the local slope is considerable groynes are typically used on the concave, or on both sides of the more slightly curved portions of the contracted channel, and on the convex side of the improved channel when the curvature is considerable; while training walls are desirable on the concave side, in the last-mentioned case, but connected with the river bank at intervals, whenever the higher waters would otherwise appear likely to flow behind the training wall with enough current to disadvantageously affect either the improved channel or the sedimentary fill behind the training wall. In any case a comparison in cost between alternative designs will be much more nearly proportional to the relative cubic contents than to comparative total length of construction, as the greater portion of groynes is in relatively shallow water, nearer the banks, while a training wall is necessarily in the deeper water for the greater part of its length. In no sort of engineering design is there greater opportunity for wise judgment in adapting the construction to the various requirements of each improvement, and in so planning that this will be done with the greatest economy and effectiveness. It is often wise to consider several alternative designs, each satisfying local conditions, and to make estimates of cost of each plan which appears to be at all closely competitive in desirability and expense.

51. Materials and Character of their Construction.—The material from which the body of the groynes or training walls is made depends mainly on what is most economically available at each locality. Stone is very generally employed in regions where it occurs, especially

in case the improvement requires rock excavation; and wood in some form usually constitutes a considerable part of the volume where rock is expensive. The hardwoods and some softwoods, as well as the heartwood of the latter variety in general, are usually very durable so long as they are entirely submerged. There is in the Fishmonger's Hall, London, a chair made from one of the piles of the Old London Bridge, built in 1176, the wood having been submerged in the Thames River for about 650 years; and the Washington University Civil Engineering Museum has several specimens of oak and red spruce piles from the famous settlement at Robenhausen of the prehistoric Lake Dwellers of Switzerland, which had undoubtedly been in water for several thousand years and are still sound and free from decay although they were comparatively soft when first taken out. Concrete has also been employed in such works, and its use is increasing; its strength and durability compensating for its greater cost, particularly in places of especial exposure.

Contracting works vary in character from those which are quite solid and so allow very little leakage, through all gradations of permeability to such as merely slacken the current and so rely upon a resulting deposit of sediment to make them effective. Permeable types are possible only in streams which are charged with silt to such an extent that the degree of retardation of velocity produced by them is sufficient to cause sedimentary action. The more impermeable kinds, of course, check the current to a greater extent; and they also become effective in rivers less heavily loaded with silt when the influence of the more permeable kinds might fail to produce results. A comparatively rapid and assured accumulation of detritus is essential because of the fragile character and light construction of the ordinary kinds of permeable works. After the new river bank has formed, it is generally necessary to protect it from erosion, especially at the channel edge, in order to hold the advantage so gained.

52. Types of Temporary and Very Permeable Works.—Various kinds of comparatively inexpensive and temporary types of permeable works have been thus employed. The "Brownlow Weed," Fig. 33,[1] is a device which was used in India as much as forty years ago; modifications of it, as employed on the Missouri River about thirty-five years ago, are shown in Figs. 34 and 35. The stiff weed, of which a cross-section is shown in Fig. 36, had its core poles over-

[1] This and the five following figures are from "Professional Memoirs," Volume IV, pp. 685–7.

lapping and firmly bound together at intervals of 3 ft. to stiffen the whole construction, thus holding in place the successive elements of brush, placed in the form of the letter "X," which followed each other along the core poles as closely as they could conveniently be packed. Such construction has been used in this country; as in the building of a traverse to close a secondary channel of the Missouri River near Fort Leavenworth.[1] It is not successful in depths of water exceeding about 6 ft. Fig. 37 (p. 192) illustrates a wire screen of large mesh, held in place by anchor stones at the bottom and buoys at the top, as employed in the same river; it can be erected very rapidly, a rate of 1000 ft. per day

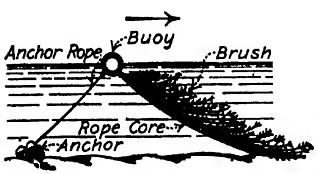


Fig. 33.

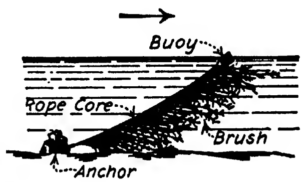


Fig. 34.

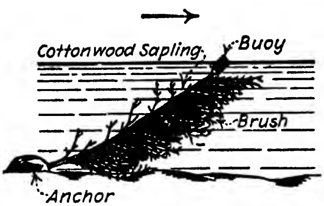


Fig. 35.

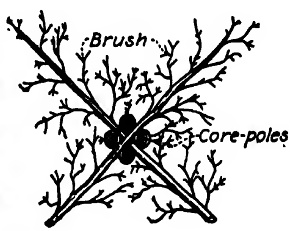


Fig. 36.

Figs. 33 to 36.—Some temporary silt arresters.

being practicable.[2] The immediate construction of permanent works of protection is often necessary to hold the advantage so gained. Another kind of screen is shown in Fig. 38 (p. 192). It was similarly supported but consisted of a curtain of willows, 1 or 2 in. in diameter at the butt, placed side by side and held in position by number thirteen wires at both the face and back of the curtain and twisted together between each piece of willow at intervals of 6 or 8 in., thus weaving the whole into a continuous screen;

[1] Report, Chief of Engineers, U. S. A., 1880, p. 1415.
[2] Report, Chief of Engineers, U. S. A., 1880, pp. 1433-4.

the space between the successive sets of wires was 4 ft. Curtains of this kind were readily made 100 ft. long and 33 ft. wide, which is a sufficient width for reaching the bottom in their inclined position in a depth of water not exceeding about 20 ft.; they are also easily rolled up as made, and then launched into place from supporting barges. Both the wire and the willow curtains produced rapid deposit in the very muddy waters of the Missouri River, bars forming in two or three weeks to a height varying from the water surface to about 2 ft. below.[1]

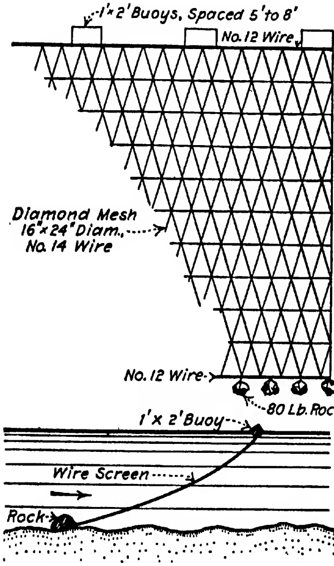


Fig. 37.—Wire screen.

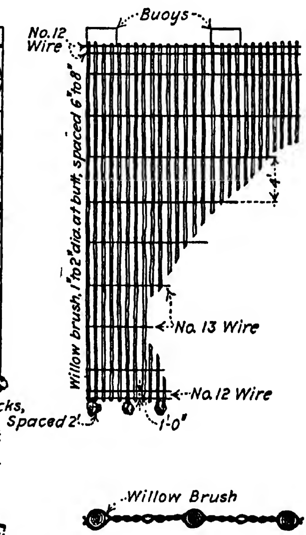


Fig. 38.—Willow curtain.

On some of the rivers of Europe a temporary and light construction of very permeable character, similar in its effects to that of the willow curtain, has been used for many decades. On the Garonne it apparently took the form of a wicker work of brush which was supported in a vertical position by rock placed along each side of its lower portion; and to induce the deposit to build itself still higher, there was employed the assistance of "flocages," or branches of green willow whose butts were forced several feet into the wicker

[1] Report, Chief of Engineers, U. S. A., 1880, pp. 1428–9.

work already in place and whose tops rose 4 or 5 ft. above it, built as compactly as the willows could be placed. The rate of

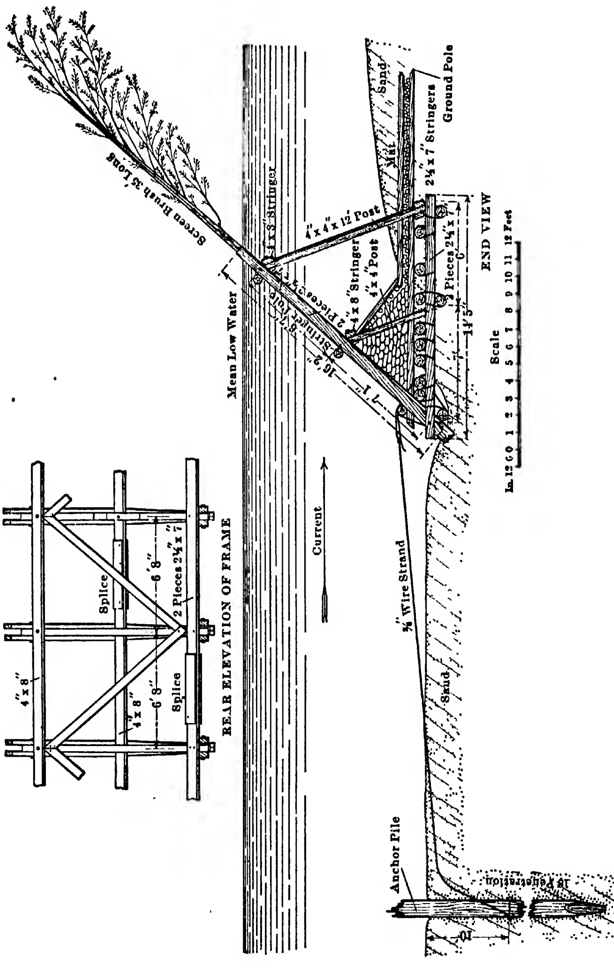


FIG. 39.—The abatis type.

accretion was much slower on these less turbid rivers, it requiring a year or two for the deposit to become complete. Then willow shoots

were planted upon the accumulated sediment in order to preserve i surface from erosion as the willow barrier disintegrated.[1]

All such very light and temporary kinds of construction, althoug comparatively inexpensive, require caution in their employment, especially in rivers of rapid current which are subject to great variations in stage of water. For these reasons their effectiveness in any project is rather problematical.

Another type of permeable works of regulation which is more permanent and more often employed is the abatis, shown in Fig. 39 (p. 193).[2] Frames of the general shape illustrated are spaced 6 or 8 ft. apart and connected with each other rigidly by timber stringers

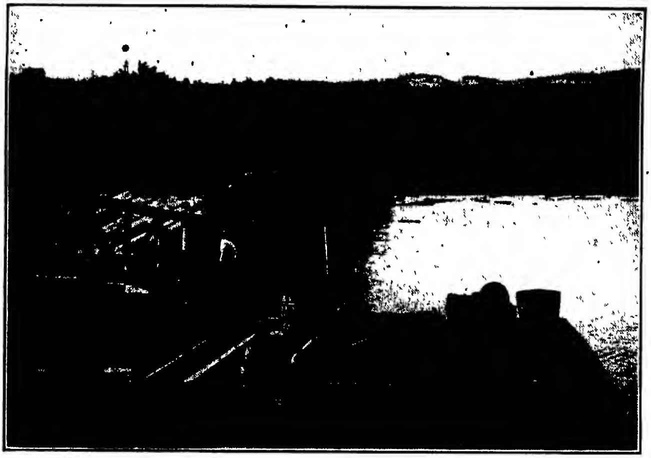


Fig. 40.—Constructing abatis groynes.

shown in cross-section. Upon the horizontal part of the structure is placed the brush mattress covered with stone to supply the needed weight which, with the anchor cables, holds the abatis in position and prevents scour of the river bed by the currents. The screen brush or poles are wired and spiked upon the inclined face of the frame in close succession, and the angle of the frame is adjusted so as to bring its upper edge to the desired level, which is often not far above the low water surface. Sections from 60 to 200 ft. in length are assembled on barges, and are launched into position on the river

[1] Brochure 4 of the Eighth International Congress of Navigation.

[2] Report, Chief of Engineers, U. S. A., 1900, p. 4779.

bed from the inclined ways provided, as illustrated in Fig. 40. The abatis is not adapted to use in currents of high velocity, nor to depths exceeding 12 or 15 ft.

53. The Employment of Hurdles.—A much more durable, adaptable, and generally effective type is the hurdle. It has been the most extensively employed of all the kinds of permeable construction. Hurdles are successfully built in any depth up to about 50 ft., although the difficulty and expense is much greater in the deeper water. A typical form of hurdle is shown in cross-section in Fig. 41.[1] Here the piles are driven about 6 ft. apart in three rows. The upper row serves the important purpose of protection against drift, etc.; the lower row gives greatly increased strength to the structure through the additional effectiveness produced by the inclined and horizontal bracing between the rows, as well as by the longitudinal

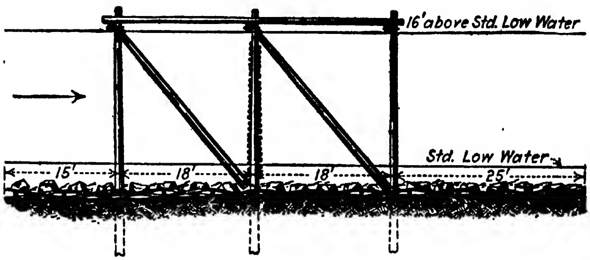


FIG. 41.—Cross-section of a hurdle.

stringers connecting the individual piles of each row. The central line of piles is here the essential one, as it carries the wattling which slackens the current and so causes the deposit of detritus from the silt-laden waters of the river. This wattling consists of small willow trees whose diameter is 2 or 3 in. at the butt; they are stripped of their larger branches, and woven horizontally about the piles, adjacent willows always being on opposite sides of each pile. The ends of the small trees overlap 6 or 8 ft. and are tied together in lengths as great as is practicable; and these can be placed about the piles above water and then pressed down to position. The weight of the workmen can accomplish this crowding of the wattling below the water surface to a depth of 7 or 8 ft. For greater depths it is necessary to use hurdling forks to push down the individual willow rods, which can be accomplished to depths of 25 ft.; or to weave

[1] Report, Chief of Engineers, U. S. A., 1885, p. 1656.

curtains of willow as already mentioned, and when the required width is finished to then attach the upper edge to the piles above water and launch the curtain from the barges with the other edge weighted so that it takes a vertical position against the piles, being held there by the strength of the current. The latter method becomes the less expensive in depths exceeding 12 or 15 ft. A mattress weighted with stone as shown in the figure, is essential to prevent the erosion of the alluvial river bed and so to safeguard

FIG. 42.—View of a finished hurdle.

the work. Adequate bank protection is also necessary to prevent the flanking of the groynes by the river currents.

The number of rows of piles may be reduced to two, or even to one (as in the Grand River, Michigan,) in localities where the velocity of current is not great, the depths inconsiderable, the fluctuations in stage moderate, or the danger from drift or ice is slight. The piles are sometimes driven in clumps of two or more, but with greater spacing between clusters, in places of considerable exposure. An example of the employment of a single row of piles only is furnished

by the works upon a section of the upper Loire, which was improved a few years ago. As the maximum velocity of current was only 5 ft. per second, it was considered feasible to secure the necessary stability by a top waling piece and a bank of large stones placed along one edge of the piles; while in the more exposed situations there were formed substantial rows of such stones on both sides of the piles but the waling was omitted.

The height above the low water plane to which such permeable construction should extend depends most intimately upon the characteristics of the stream. Because the particular purpose is to secure a deposition of sediment so rapid that it will be complete before these comparatively temporary kinds of works are destroyed, they should reach to that stage of the river which carries a large amount of silt. Yet the height which is desirable for that reason may involve too great danger from drift, ice, or severe current action; or may so much restrict the area of cross-section that the flood level will be seriously raised. All these factors, with that of cost which increases with height, must be considered in a conclusion which will secure the maximum aggregate advantage. In the hurdle illustrated in Fig. 42 the construction reaches about 16 ft. above the low water stage; and instead of a wattling woven horizontally about the middle row of piles, screen poles are placed almost vertical and spiked to the down stream line of longitudinal braces.

Hurdles have been used most extensively on the Mississippi River. In order to illustrate the development of their adaptation to a particular case and to indicate the progressive character which usually marks such operations when extensive, the experiences of the last forty years at Horsetail Bar will be outlined. The name is applied to that portion of the Mississippi extending for a distance of about 5 miles from the mouth of the Des Peres River at the southern end of the city of St. Louis to the foot of Carroll's Island. Previous to the beginning of work to improve the channel in 1873, this stretch of river averaged nearly a mile in width and its bed was composed of a silty and sandy sediment which made an extremely unstable bottom. These facts, together with the practically straight outline of the river here, the great amount of alluvium contributed by the Missouri River hardly 30 miles above, and the considerable velocities especially at high stages, all combined to produce a stretch which was notorious for the difficulties of its navigation. In the distance of 5 miles there were several bars whose positions were constantly changing, and a navigable depth at low water of only 4 ft.

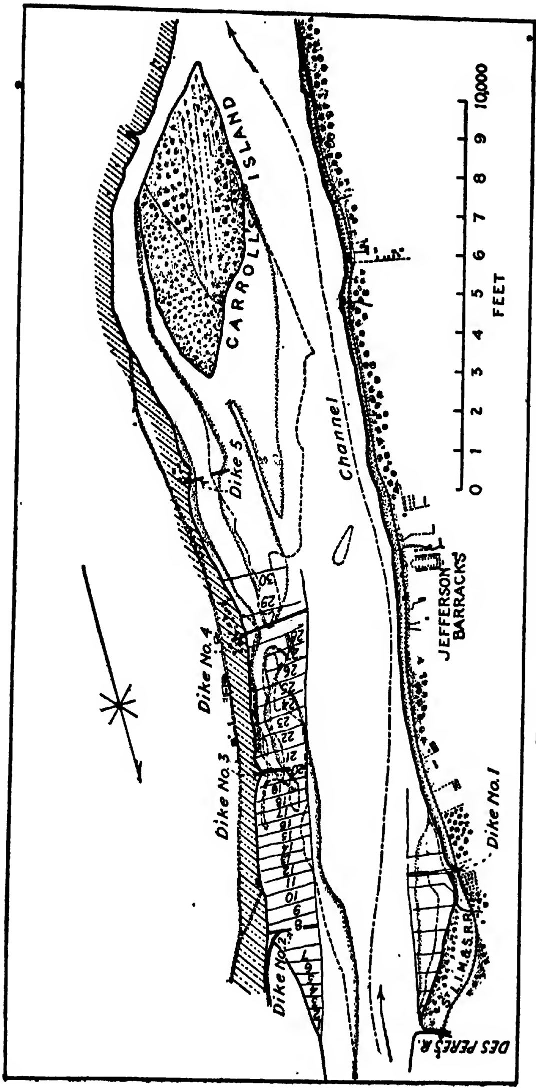


FIG. 43.—Horsetail Bar, Mississippi River.

was not unusual. The river as it then existed is represented approximately by the heavy full lines of Fig. 43,[1] with nothing but open water between.

The project under which work was begun in 1873 contemplated reducing the channel width to 2500 feet by groynes ("jetties"); one extending from the Missouri bank in the sharp concavity near the upper end in the position indicated as "Dike 1" of the preceding figure, and four from the Illinois bank at intervals of about 4000 ft., as indicated by "Dikes 2, 3, 4, and 5" of the same figure. The channel thus outlined would be but slightly curved, and it was hoped that the reduction of width would substantially improve the navigable depth. The material employed in these first contracting works was mainly a foundation of brush from 2 to 5 ft. thick and of a width sufficient to accommodate the mass of stone riprap rising at its natural slope to an elevation of 8 ft. above low water; an apron of brush and stone was constructed along the down stream edge of each groyne to prevent its undermining by the fall of water over it; piles were also partly used to aid in holding the construction in position, and in places other minor modifications were made. During the six years of work under this plan an aggregate length of about 8300 ft. was completed in these five groynes.

The depth of water in the channel in 1874, as a result of the construction of Nos. 1 and 3 and the building of groyne No. 4 for more than half the proposed distance from the left bank, was stated to be considerably greater than that of the year before at the same stage, the particularly troublesome crossing having deepened 2 ft. However, the next year disclosed the formation of a serious bar where satisfactory conditions had previously existed, about a mile and a half above the location of the previous difficulty; consequently, No. 2 was built to a length of 1300 ft. There also occurred an extensive deposit of sediment, at the higher stages, not only between the groynes but filling "the whole bed of the river to a height of 8 or 10 ft. above the normal low water plane." As the stage became lower a shifting and difficult sailing channel was formed which lay close to the exposed end of No. 1 at the right bank, then crossed to the left shore and followed it until diverted by No. 3 back to the right bank. Groyne No. 5, which had been partly constructed to close the secondary channel on the left of Carroll's Island, was partly buried in the deposit, but in one place was undermined and breached. Thus a volume of sediment accumulated between

[1] Plate 4 of Report of Chief of Engineers, U. S. A., 1882, p. 1654.

and about the groynes, but lacked stability; a marked indication of this was the fact that the only practicable sailing channel broke through No. 1 near mid-length. A considerable settlement of portions of the groynes was also noted. The seriously wandering character of the channel when the river was falling from a mean stage was thought to indicate a deficiency in height of the groynes, and they were consequently raised to an elevation of about 14 ft. above low water in the years 1876 and 1877, with the result of a general improvement in the channel at the time. A more positive control of the flow at lower stages than could be possible with groynes spaced three-quarters of a mile apart was proposed in the construction of a training wall connecting their ends; and this part of the original plan was carried out by the building of a length of 8150 ft. to a height of 12 ft. above low water in the years 1877

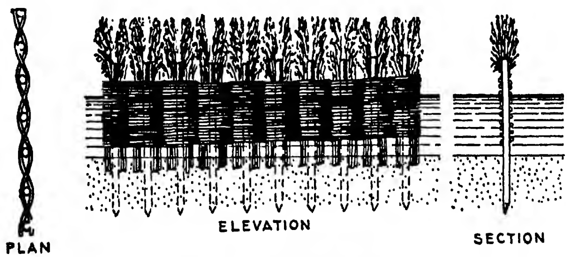


FIG. 44.—A completed hurdle.

and 1878. However, neither a permanent and effective contraction of channel width nor the permanence of the works was effected. Breaches of the groynes became more numerous and extensive, rather than less so; one of the worst of these was 300 ft. wide and 45 ft. deep at its deepest part, and the surge of water through it formed a hole 70 ft. deep just below the break. The great weight of this type of construction caused settlement in many cases, with the consequent expense of repairs and severe disturbance of régime; in one case the sinking amounted to 28 ft. These recurring and increasing difficulties were serious and the expense for repairs became so great that the conclusion was reached that this heavy, solid and expensive type of construction was not suited to a river of such volume, velocity of current and range of stage as that of the middle Mississippi, especially when accompanied by so unstable and treacherous a soil forming its bed. Accordingly, the dilapi-

dated stone and brush construction was wholly abandoned and hurdles were substituted in 1879.

Under the new plan hurdles were begun in that part of the river lying between the old training wall and the left bank, which was to be reclaimed. These first groynes were spaced about 300 ft. apart and consisted of a single row of piles driven at intervals of 5 ft. about which the wattling of willows was woven to a height of 15 ft. above low water, with similar pieces of brush crowded vertically into the spaces remaining at the sides of the piles so that their butts would penetrate the mud at the bottom and their tops would project 5 or 6 ft. above the upper edge of the wattling, all as indicated in Fig. 44.[1] Eight groynes were constructed the first spring before a high water occurred, and when it subsided the whole space between them was found to be filled with sediment as high as the top of the wattling. This experience was so promising that construction was continued dur-

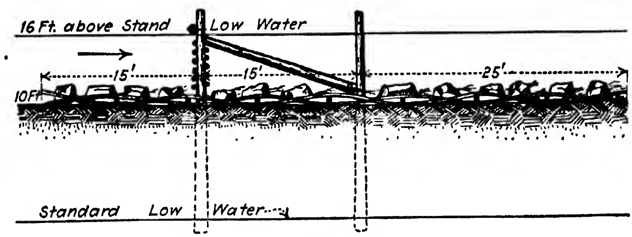


FIG. 45.—A hurdle for moderate depths.

ing the years immediately following. In 1882 twenty-nine groynes had been built on the left bank and nine on the right, as indicated on the map of Horsetail Bar already shown. Extensive additions on the line of the old stone training wall were also necessary. Different kinds of structures were tried, among which were curtains held in position by various methods; but it was finally found that hurdles were most effective in this longitudinal construction also. In arriving at these results cost, effectiveness, and durability were all considered. The hurdles are so comparatively light that they do not settle into the soft alluvium of the bed; their permeability allows the silt-laden waters to penetrate them readily, with a minimum of disturbance, and yet so slackens the current that extensive deposit results; while they are not proof against damage by severe flood conditions, excessive ice pressure, etc., other types of contracting works are also

[1] Report, Chief of Engineers, U. S. A., 1879, p. 1028.

injured by such serious occurrences and the hurdles are very easily, quickly and cheaply repaired. The details of their construction are readily adaptable to the degree of severity of adverse conditions existing, unless it be excessive, by increasing the number of rows of piles or the thoroughness of the bracing, or by using clusters of piles with their tops strongly and closely drawn together by wire rope in the more exposed situations.

Experience finally determined that it was generally best to limit the use of hurdles consisting of a single row of piles to depths of water not greater than about 3 ft. For depths between 3 and 6 ft., two rows of piles were found desirable, especially in a considerable current, as shown in Fig. 45 (p. 201).[1] For ordinary depths the standard construction took the form shown in Fig. 46; and in more than 18 ft. of water or in particularly swift currents the more thoroughly braced type was used, as illustrated in Fig. 41 at the beginning of the discussion of hurdles. In case construction is carried on when the river is above a stage of 16 ft., the top of the hurdles must of course be high enough to extend somewhat above the existing stage. The large clevis used for making the difficult connection under water at the foot of the inclined braces is shown in Fig. 47 (p. 204). Protecting mattresses, as shown, are essential in every case; and at the bank a revetment was generally found necessary for a distance of about 100 ft. both above and below the ends of the groynes, and extending to the top of the bank. The hurdles were constructed from the shore end outward, and the piles should, to secure permanency, penetrate a distance as great as the depth of water, and in any case not less than 10 ft. The standard spacing of the piles was 6 ft. in the middle row, and they were placed twice this distance apart in the two outside rows. When a hurdle is built in water exceeding 15 ft. in depth the wattling may advantageously be raised to only about half that height at first, the finishing of this work being accomplished after the sedimentation has reached that half height. A view of a completed hurdle is shown in elevation in Fig. 48, (p. 204) and in plan, with protecting mattress on river bottom, in Fig. 49 (p. 205)

The progressive character of the formation of the deposit was quite satisfactory. Occasional breaches in the structures allowed currents to scour out incipient channels, but their repair would cause the sedimentation to again proceed. The water-courses and pockets which had persisted under the former type of construction

[1] This and the three following figures are from Report, Chief of Engineers, U. S. A., 1885, p. 1656.

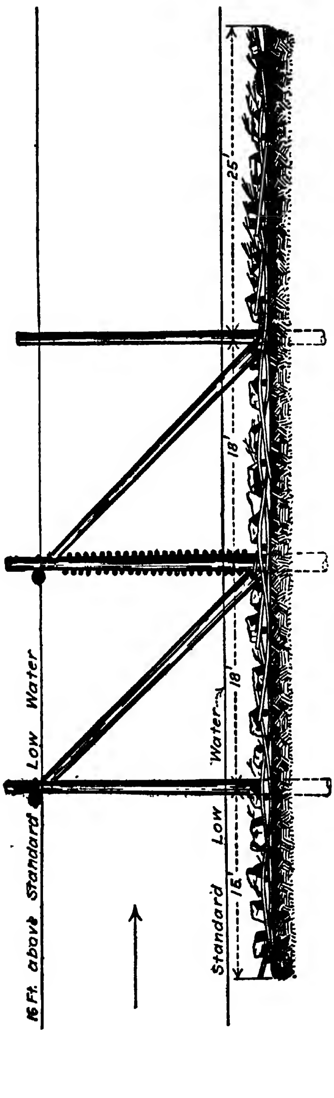


FIG. 46.—Standard form of hurdle.

were largely eliminated; in one place where there had been 60 ft. of water a deposit formed which was 80 ft. deep. In two years the quantity of material induced to form among the hurdles amounted to more than 13,000,000 cu. yd. As the deposit reached about 15 ft. above low water, the willows began to grow upon the surface, which

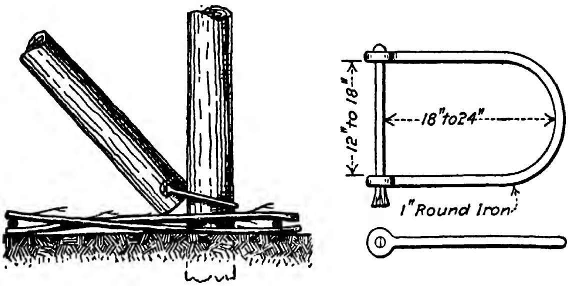


FIG. 47.—Clevis at foot of braces.

in turn caused a continuation of deposit that attained a height for considerable areas of 20 to 30 ft. above low water. These favorable results permitted, in 1881, the cessation of the construction of hurdles at the close spacing originally planned because the influence of the work already done had reached as far down-stream as Carroll's Island,

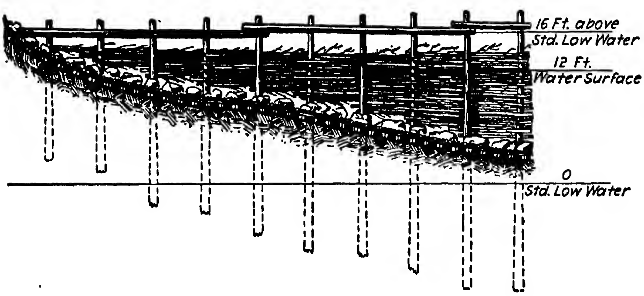


FIG. 48.—Elevation of a hurdle.

and thereafter hurdles were built only at places where inadequate development of the accretions showed their need. Consequently the prolongation of the sedimentation farther down-stream was induced, during the next several years, by the construction of occasional groynes, by lengthening the training wall and by the more

complete closing of the chute on the east of Carroll's Island, toward which the currents continued to set strongly. In 1893 the deposit

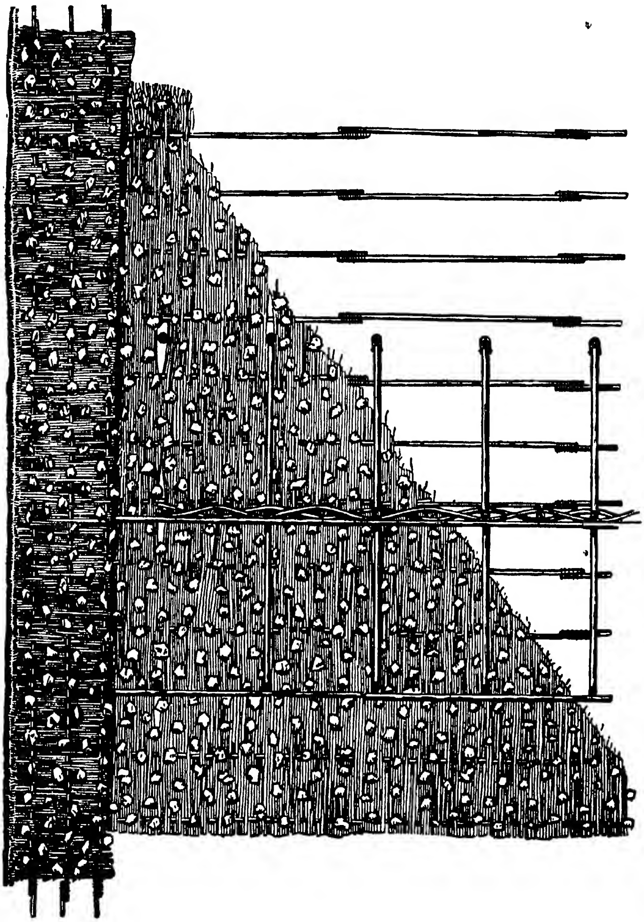


FIG. 49.—General plan of hurdle.

had extended southward to the foot of Carroll's Island, filling the old bed of the river from the Illinois bank outward to a general line extending from the training wall to the front of that island.

In 1884 the accretions produced by the hurdles on the right bank

had reached a height of 25 ft. above standard low water and the top was protected by a vigorous growth of willows. The exposed channel edge was found to be gradually eroding under the attacks of the current, and it therefore became necessary to protect this face as had been anticipated. A mattress was therefore constructed here, 100 ft. in width and 3880 ft. long, and rip rap was placed on the bank above it. This revetment, both above and below low water, was widened and extended at various times, particularly in 1893 and 1894 when it was completed for the full extent of the induced deposit, protecting the exposed edge for a total of about 1½ miles below the mouth of the Des Peres River. A similar protection of the channel edge of the extensive fill along the left bank was found unnecessary except in the vicinity of Carroll's Island, because a seriously eroding current did not reach it elsewhere.

The success of the hurdles in producing extensive bank building to reduce excessive widths in this river is indicated by the fact that, in 1888, the made ground at Horestail Bar covered an area of 915 acres to an average depth of almost 12 ft., or about 17,500,000 cu. yd.; of this, an area of 589 acres was above the stage of 15 ft. and was covered with a thick growth of willows. In the next few years the accretion was increased and extended down-stream over large additional areas, but neither the acreage nor the volume is given. The transformed condition of this portion of the river is indicated by Fig. 50.[1] While high floods still submerge this artificially formed area, a large portion of it is above ordinary stages of the river and much of it is now under cultivation.

The effects upon the channel are so extensive and satisfactory that what was naturally considered the worst shoal place below St. Louis now gives no trouble. A regular and definite sailing course through Horsetail Bar has been established by the works of improvement, which has had for a score of years the projected navigable depth of not less than 8 ft. at all stages.

The total cost of this improvement from 1873 to 1912 has been $1,097,294, of which more than one-fifth was for the abandoned stone and brush construction of the earlier years. The bank protection cost somewhat more than 8 percent of the total, and the hurdles, about 70 percent.

The successful results attending the employment of hurdles at Horsetail Bar resulted in their frequent construction at other places in the middle and lower parts of the Mississippi River. Their use

[1] From Report, Chief of Engineers, U. S. A., 1895, p. 2090.

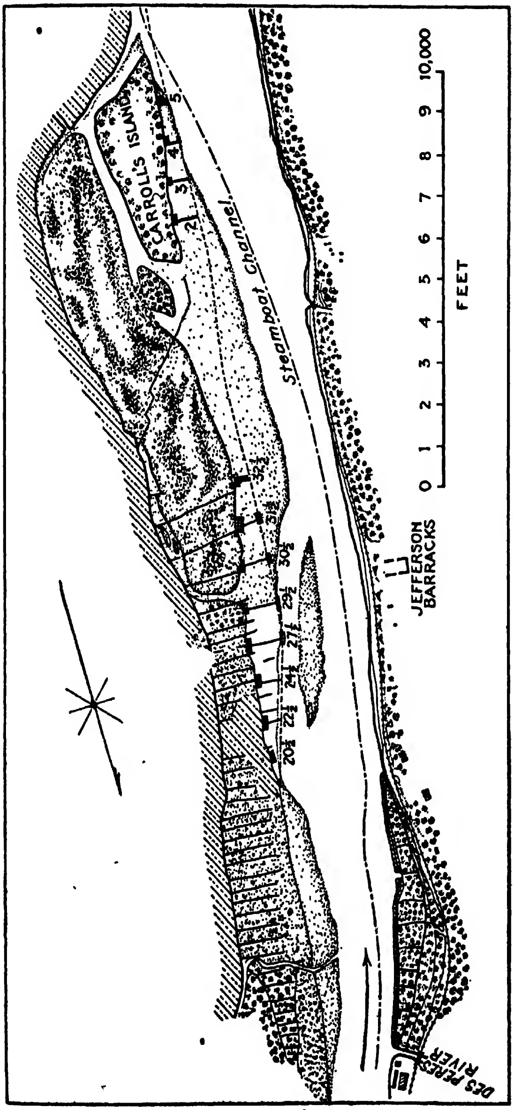


Fig. 50.—Horsetail Bar, after improvement.

has been especially extensive in the 190 miles between the mouths of the Missouri and Ohio Rivers, where 60 miles of bank has been protected by revetment and 71 miles of channel was contracted by hurdles, in the thirty years following their first trial as noted. While much work remains to be done in thus artificially extending the banks to contract this part of the river to a width of 2500 ft., the work so far done has greatly improved its navigability. In case a greater depth should be desirable, a further contraction at the low water plane would be planned; for example, the suggested deepening to 14 ft., by regulation only, involves a low water width of channel of probably 500 ft. This system of a moderate contraction for

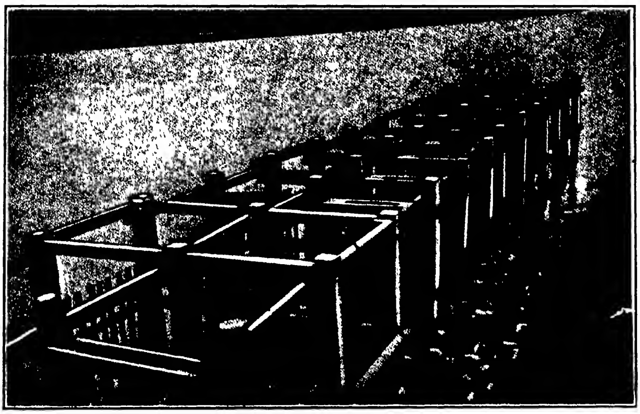


FIG. 51.—A reinforced concrete hurdle.

a medium stage and an additional contraction for low water is in entire harmony with many European experiences, as on the Loire, "to train the waters during the whole of the time that there is any carrying away of the materials of the river bed."[1]

The fact that such hurdles are found by experience in this country to have a life of only eight or ten years has recently led to the construction of the framework and bracing of reinforced concrete piles and beams in cases of especial exposure, as where the danger from ice pressure or gorges is considerable, or where it is doubtful that sedimentation will protect them before they decay, which may occur in portions of training walls and the exposed ends of groynes. The

[1] Brochure 6 of the Twelfth International Congress of Navigation.

less durable wattling or curtain may then be replaced, when necessary, at slight expense. The first instance of the use of reinforced concrete piles and frame for the outer end of a groyne seems to have been in 1908 on the Missouri River near St. Joseph, and illustrated in Fig. 51,[1] in which the concrete construction is shown extending from the wooden framework at the left; and vertical screen poles, wired to the longitudinal braces, are used instead of wattling. One of the examples of reinforced concrete in training walls is that which was built the next year on the Republican River at Fort Riley, Kansas. In this smaller stream two rows of piles were considered sufficient, the screen poles being attached to the bracing of the landward row. The spacing of the piles, as well as the distance between rows, was about 10 ft. The concrete used throughout was a 1:2:4 mix of Portland cement, river sand and limestone. The piles were about 11 in. square and 30 ft. long, reinforced with four longitudinal bars of $\frac{3}{4}$ in. diameter and with $\frac{1}{2}$-in. steel hoops spaced 2 ft. apart. The struts were 8 in. square, each having four steel bars of $\frac{3}{8}$ in. diameter placed near the four edges; they were molded in forms built in place after the piles had been driven. The construction and sinking of the mattress, 40 ft. in width, as usual preceded the driving of the piles. Later construction of this kind has shown some improved details, especially in the connections employed for the bracing on more exposed and heavier work, such as the leaving of holes, when molding the piles, through which the reinforcement of the struts is later extended.

The construction of hurdles has long been practised in Europe as on the Garonne for the years 1836–1846, inclusive.[2] The piles were of pine about 8 in. in diameter. They were driven 7 or 8 ft. into the river bed; and were of such length, depending upon the depth of water, that they projected from 4 to 8 ft. above the low water surface. They were spaced somewhat more than 4 ft. apart, and wattling was woven about them and pressed down to at least 3 ft. below the water surface, in the manner already described. The wattling also extended to the tops of the piles; and then vertical branches ("flocages") were crowded into the hurdles, their length being such that their tops were about 10 ft. above the tops of the piles, in order to encourage the deposit of sediment. Rock was placed about the foot of this construction, resting upon a fascine base wherever necessary, in order to

[1] Report, Chief of Engineers, U. S. A., 1909, p. 1661.

[2] Annales des Ponts et Chaussées, 1848 (2), pp. 1–157.

aid in its support, to fill the space below the reach of the wattling and to reduce the scouring effect of the currents, a danger which was only moderate as the soil was generally a gravelly one. Occasionally when the current velocities were high, a double line of this sort was constructed; the two lines were placed 6 ft. apart and well braced, and the space between was filled with gravel resting upon a mattress of fascines.

54. Compact Kinds of Structures Used in Lateral Contraction.—Comparatively impermeable works of contraction are often made by using two or three rows of braced wooden piles for the framework, but forming the barrier to the current of a much more compact construction. An illustration of this kind is shown in Fig. 52,[1] in which there are alternate layers of fascines and stone packed solidly between the two upper rows of piles which are tied together by a tension rod to resist the thrust caused by the packing. Frequently only stone is employed above the low water surface, so as to avoid the early deterioration resulting from the decay of wood. Other materials are also used in a similar way when more available; such as logs instead of fascines, gravel or sand in sacks or boxes instead of stone, or even gravel or sand only when a support is furnished by a sheathing which is carried by the rows of piles. Solid pile groynes and training walls are practicable only in rivers which have a bed of sufficient resistance to safely carry the rather heavy load occurring in this type; there have been many instances of a serious settlement of the material placed between the rows. Sometimes the supporting framework takes the form of a crib, especially in a river with a gravelly or rocky bed in which the driving of piles would be difficult or impossible, as illustrated in Fig. 53.

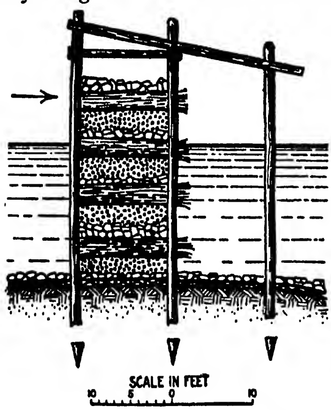


FIG. 52.—A more impermeable type.

It is very often the case that the saving of material and other advantages do not compensate for the extra expense of the pile

[1] This and the following figure are from Brochure 39 of the Twelfth International Congress of Navigation.

or crib support; and a more extensively used type is that in which the materials are placed in a much broader construction with inclined sides at least as flat as the slope of repose of the material used, the larger mass being depended on to secure stability. Fig. 54 (p. 212) illustrates a groyne of this kind, consisting of alternate layers of fascines and stone, which has been often used. It will be noticed that the layers dip considerably up-stream in order to better resist the effect of the current. Fascines form a large part of such construction because of the rapid and widespread growth of the willow and other similar brush, of which they are made, in alluvial bottom lands where other materials are usually much more expensive; and because of the compact and yet pliable form thus secured, which makes them effective in their final position.

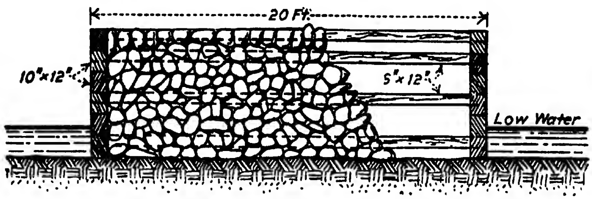


FIG. 53.—Section of a crib.

Contraction works of fascines and stone are very common. On the upper Mississippi River, for example, groynes are often constructed upon the timber ways of a barge devoted to this purpose, which is moored in a position transverse to the current and with its lower side about on the line of the upper edge of the groyne. The brush and poles are passed to the building barge from other barges brought alongside, and all are held in position by lines running up-stream to anchors or piles, and also to the bank. Depending on the desired width of mattress, two or three lines of binding poles are laid upon the tilting ways parallel to the side of the building barge, and transversely upon these are placed the bundles of brush with the butt ends pointing down-stream. On top of these bundles are then laid a second series of binding poles which are firmly tied through the brush to the poles beneath at frequent intervals by No. 16 galvanized wires. The mattress thus compacted and completed is launched by the raising of the inner ends of the supporting timbers until it slides down-stream into the water,

and its movement during launching and subsequent sinking is controlled by mat-lines tied to the up-stream set of binding poles. The rock for the sinking is thrown upon the mattress from the stone barge which is generally placed 15 to 30 ft. below the building barge, the mattress being therefore held between the two barges after launching. Skill and experience in manipulating the mat-lines and in the throwing of the rock upon it generally secure a quite satisfactory sinking of the mattress into the desired position. The lowest mattress is made 10 or 15 ft. wider than the one next above it in order to protect the groyne effectively from being undermined by the currents, especially when the waters sweep over the top and fall with a scouring action at the down-stream edge of the structure. This foundation mattress is also weighted with an extra amount of rock, and the suc-

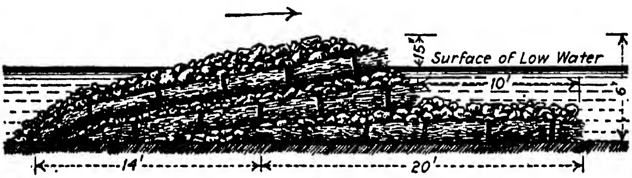


FIG. 54.—Fascine and stone construction.

cessive sections of it are completed before beginning the next layer above. Then additional mattresses of greater thickness, but successively narrowing toward the top, are launched and sunk until the groyne is complete. The total volume of brush in these structures has varied from one to three times that of the stone used. A frequent modification in procedure is the construction of the lower mattresses on another barge called a "grasshopper," which is provided with the necessary equipment and is stationed between the building boat and the stone barge.

Groynes and training walls are frequently made of rock only, in localities where the river bed is firm enough to carry the heavy load either with or without the aid of a mattress, especially when the stone is available by reason of the necessary excavation of a nearby channel in rock, or under other conditions which make it an economical material. Fig. 55[1] shows a stone groyne, with its woven mattress to resist scour and settlement in a quite firm but somewhat erodible river bed. In case a greater degree of water tightness is desirable,

[1] Report, Chief of Engineers, U. S. A., 1902, p. 1576.

gravel or even gravel and clay can advantageously be used in filling the joints of the rubble. On the Rhone, the stone groynes were built in stages to encourage a more rapid and thorough sedimentary accretion to take place; and on placing the portion extending above low water the voids of the rubble were filled solidly with concrete. In Europe gravel has sometimes been used, wholly or in part. Fig. 56 is a cross-section of a training wall formed of a core of gravel covered by layers of stone, all supported by a foundation mattress.

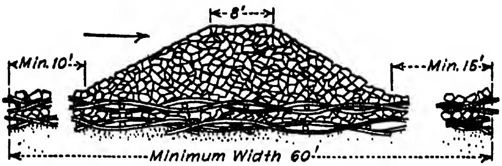


FIG. 55.—Section of a stone groyne.

Economy in cost has also sometimes led, both abroad and in this country, to the employment of a core of sand, or other earth, protected against disintegration by a covering of fascines or fascine mattresses loaded with stone, gravel, or even with brick rubble.

It must be understood that the foregoing illustrations of construction are merely typical. Very many variations and modifications occur in different works, such as poles or even timber in the abatis type; hurdles with mattress construction inclining from the river

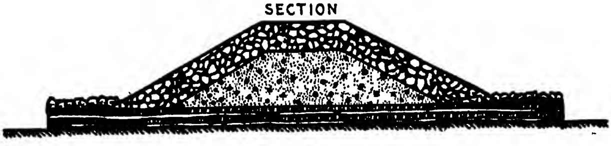


FIG. 56.—Training wall with gravel core.

bottom to the tops of the piles, or other styles of curtain instead of the ones shown; pile construction in which the bracing is omitted but the layers of fascines and stone extend beyond one or more rows of piles as well as filling the space between; timber mattresses instead of brush or fascine construction, etc. Good engineering always adapts the dimensions to the conditions of service, and adopts those materials which are most economical in regard to cost and the requirements of the work. It is believed that the illustrations given will

convey a general understanding of the principal types more usually employed.

55. Details of Design, Construction and Cost.—Whenever contracting works are built upon soil that is at all unstable or yielding, their permanence requires that they shall rest upon a bottom mattress of fascines or other protective construction in order to preserve the integrity of the works at the base, especially by its projecting so far beyond their edges that any scour by the current shall not endanger their stability. It is thus necessary to guard against erosion, not only from the severe horizontal currents which always exist along the margins and ends of obstructions in a stream, but also against the scour that results from the fall of water over their tops, which occurs in the more solid types of construction at stages somewhat higher than the crest of the works. The latter case necessitates an especial width and resistance at the down-stream margin of the structure. Methods used in dealing with these requirements are also illustrated in preceding figures. Similar careful attention, both above and below low water, to protection from erosion is required at the bank end of all such works, where stone rip-rap or paving or concrete construction should be used above the lowest water surface; and particular thoroughness in this regard is necessary at the channel ends of groynes and at both sides of training walls. In cases where contracting works are built entirely on the convex side of the channel, thus exposing the concave bank to the action of the accelerated current, bank protection should be constructed if the velocity is an erodible one at any stage.

The transverse sections of groynes and training walls necessarily vary as the materials and the conditions at the site of their construction differ. Some such dimensions have already been indicated on figures illustrating various kinds of contracting works. On the Waal the groynes, constructed of a core of sand protected by fascine mattresses weighted with stone, etc., have a top width of about 11 ft., side slopes of 2 on 3, and a slope at the channel end of 1 on 4; the thickness of the top layer of rubble is 20 in. In the Weser the finished slope of the channel ends varies from 1 on 5 to 1 on 10 on both sides of gently curving bends; but on sharply curving portions the end slopes are adjusted unequally in order to more nearly conform to slopes taken by natural river banks, yet so that the sum of the two slopes will equal the constant value which both would have in a slightly curving river; thus where the gently curving stretches have end slopes of 1 on 7, a sharp

bend may have its groynes finished with end slopes of 1 on 3 in the concave side and of 1 on 11 on the convex side of the channel in order to better adapt the construction to the natural regimen. On the Ohio crib groynes were about 20 ft. wide and had, of course, vertical sides. On the Allegheny River the rubble works are sometimes 40 ft. wide at the base, the side slopes taking the natural slope of the material, about 1 on 1. On the upper Tennessee River the contracting works are constructed of rock with a top width of about 8 ft. for the groynes and 10 ft. for the training walls, as a rule; particularly exposed portions are strengthened by increasing their dimensions up to double those given, or more; the side slopes are about 1 on 1. The channel ends of groynes of fascines and stone on the upper Mississippi are built with end slopes of about 1 on 1; the slope of the up-stream edge is about 1 on 2, and that of the down-stream is about 1 on 1. On the Willamette River a combination of logs and stone, placed about two rows of piles, is used for the contracting works; at the top the width is equal to the distance between the rows of piles, or 5 ft., and the bottom width is 30 ft. The stone groynes of the Garonne are finished with side slopes of 1 on 1. Apparently as yet there has been but slight attention given to the question of minimizing the disturbing effect of the groynes upon the flowing currents. Some European experiments made to determine this feature of design developed the fact that the most favorable results were obtained when the longitudinal crown line had the form of a continuous concave curve, and the cross-section of the groyne had a convexly curved upper surface merging smoothly into side slopes which were both concave in form. Flat surfaces meeting at angles always produced considerable disturbances.[1]

For reasons of economy in construction and the avoidance of measurably raising the crests of floods, it is usual to fix the height of the top of works of contraction at or somewhat above the low water surface of the river. Thus on the upper Loire the top of the contracting works was placed at the low water surface at points of reversed curvature, remaining at this level on the convex side of the channel but gradually rising on the concave side toward the middle of the longitudinal marginal curves where the height reached 40 in. above low water. Those of the Danube, Rhine and Elbe rise but slightly above the low water level. On the upper Tennessee it has

[1] Zeitschrift des Oesterreichischen Ingenieur und Architekten-Vereines, 1902, p. 469.

been the practice to raise them to about 2 ft. above low water. On the Lower Rhine, Lek, Weser, Waal and Yssel the elevation of the crest is about 20 in. above the ordinary low water surface at the channel, thence sloping upward toward the bank at a rate of 1 on 100 or more. Similar works on the Garonne were first built to a height of about 3 ft., but were successively raised to expedite the accumulation of sediment until they finally reached an elevation of about 10 ft. above low water. The groynes and training walls of the Rhone, Meuse, Ohio and Willamette Rivers have been built to an elevation of about 4 ft. above the ordinary low water level; while those of the upper Mississippi have generally varied between that height and one about 50 percent greater. On the contrary it has been the custom on the middle and lower portions of the Mississippi River to build such works to about 20 ft. above low water in order to reduce the excessive width at ordinary stages so as to more definitely utilize the current in forming the low water channel, as well as to safeguard the exposed part of the works to a greater extent against the dangers of ice and drift which occur more frequently at stages of less than 20 ft.; the great width of the river here renders the resulting reduction in cross-section of high water flow comparatively unimportant, and the results of extended experience have apparently justified their construction to this height, so much greater than is usual in rivers of ordinary size. On the Missouri, similar works have usually extended 10 or 15 ft. above the low water surface; but the general plan of improvement of 1910, for the lower part of this river, determined that contracting works should be as low as convenience of construction would allow because of the resulting reduction in cost and in exposure to injury, and the diminished obstruction of the channel to flow at the higher stages.

The direction given to groynes has also varied. On the rivers of Holland they were formerly planned to extend normal to the current, but in recent years the practice has been to point them up-stream at the channel end so that they form an angle from the normal of 10 degrees on the concave side and of 20 degrees on the convex side of the channel in order to minimize the disturbance to the axial direction of the currents and so favor the deposit of silt between them. In Germany and on the upper Mississippi River they are usually built to incline up-stream from a normal direction 15 or 20 degrees in straight portions of the contracted channel, 10 or 12 degrees at the convex side of curved portions, and 10 degrees or less on the concave side; sometimes the alignment and spacing of the groynes on opposite

sides is so fixed that their axes shall intersect in the middle of the channel.

Concerning the advantageous spacing of groynes there seems to be no formulated rule. Undoubtedly they must be closer together in places of high velocity of current than in those where the rate of fall is moderate; and they also require a closer spacing when short than when long in order to secure an equivalent degree of protection to the bank and inducement to the deposit of sediment between them whenever these are important. A false economy has frequently caused too great a spacing, with the subsequent necessity for the construction of training walls connecting their ends, or for a doubling of their number by the construction of intermediate groynes, or occasionally of both these modifications. In the Waal their distance apart has varied from about 500 to 660 ft.; in the Lek and the Rhine in Holland, from 330 to 500 ft.; in the Yssel the maximum spacing is 330 ft. In general it may be said that European practice fixes their distance apart somewhere between the width of the contracted channel and half this width; for example, on the Garonne, the average spacing was little more than half the channel width, but their frequency was doubled in currents of unusual velocity. On the upper Mississippi and in Germany the spacing is typically about half the channel width on the concave margin, perhaps seven-tenths of this in straight portions, and approximately equal to the width of the contracted channel on the convex side. For the suggested improvement of the middle portion of the Mississippi River to secure a navigable depth of 14 ft. at low water, the spacing as proposed varies from about one-third the width of the contracted low water channel to about three times its width, according to local conditions, and averages a trifle less than the mean width.

Some unit costs of different types of works of lateral contraction, as given in various published statements, are as follows: On the upper Tennessee, where they are built of stone, the cost averages about $2 per cubic yard; at the Little River Shoals, where they were mainly constructed from material excavated from the channel in order to deepen it, their cost was $2.55 per cubic yard, or an average of $1.74 per lineal foot, where rock excavation cost $1.92 per cubic yard ($1.71 for drilling and blasting and $0.21 for removal), and gravel cost $0.10 per cubic yard to excavate. During the first thirty-five years of the construction of fascine and stone groynes and training walls on the upper Mississippi River a total volume of nearly 9,000,000 cu. yd. of rock and brush was used in average propor-

tions of about two of the former to three of the latter; the mean cost was $0.84 per cubic yard, or about $6 per foot of length. Contracts of 1911, for nearly 400,000 cu. yd. of the same materials used in similar proportions, averaged $1.04 per cubic yard in the structures; the price of stone was $1.61, and that of brush in place was $0.66 per cubic yard. On the Russian river Don the average cost of 5500 ft. of training walls and groynes, made of fascines weighted with rock, was $7.89 per lineal foot. The quite temporary bank building device of wire screens, as used to some extent on the Missouri River with its buoy supports and stone anchors, has been placed at costs varying from $0.25 to $0.50 per foot of length. Abatis construction in 1899 at Point Pleasant, on the lower Mississippi, cost $4.38 per linear foot for a total length of nearly 9000 ft.; this cost was relatively high because of the unusual difficulties at the place, the velocity of the current exceeding 6 ft. per second and the depth of water being from 22 to 30 ft. As hurdles are adaptable to both deep and shallow waters, their expense has a correspondingly large variation. For example, on the comparatively small Garonne River their cost, for an aggregate length of more than 13 miles, averaged $4.96 per linear foot. On the contrary, for the middle and lower Mississippi River, where much of the construction has been in water of considerable depths, the first cost of hurdles has ranged from about $4 per linear foot to six or eight times that figure; the total outlay, including maintenance and renewal upon an aggregate length of more than 77 miles constructed on this river between the mouths of the Missouri and the Ohio, from the year 1879 to 1912, has averaged $17.92 per linear foot built. In recent years hurdles of concrete reinforced by steel have been considerably employed instead of wooden piles and bracing because of its greater durability and strength, although the cost is increased about 50 percent. Fifteen hundred feet of such construction on the Republican River at Fort Riley in 1909–10 consisted of reinforced concrete piles 30 ft. long placed at intervals of 10 ft. in two rows 10 ft. apart and braced with the same material at two levels, with the usual foot-mattress 40 ft. wide, screening poles, etc.; it cost $13.03 per lineal foot. A similar piece of work on the Missouri River near St. Joseph in 1910, but consisting of three-row work in a more exposed position, where the length of piles varied from 35 to 50 ft., involved an outlay of $16,586 for 500 ft. of length, or an average of $33.17 per lineal foot. It is thought that the expense of similar construction in the future will be about $25 per foot of

length, complete. Unfortunately, reports of costs often do not give the depths of water and other important general conditions under which construction occurred so that a more definite apprehension of their actual expense may be obtained; but the figures given will convey a general impression of the relative costs of the different types. One record of the charge for maintenance states that the expenditure during fourteen years for this purpose on the rubble stone groynes and training walls of the Rhone, built during that time, and also on the older works constructed a score of years before, was about 6 percent of the first cost for this whole term of years. The annual cost for the maintenance of the contracting works of the Weser, consisting of stone, gravel and fascines, has been estimated at about 1 percent of their first cost. In general it may be said that the expense of maintenance appears to average about 1 percent per year, varying from perhaps one-third to several times that amount, depending upon the character of the materials used and the adequacy of the design and construction to meet the required service.

56. The General Effects on the River and Its Bed.—The general tendency of works of lateral contraction upon the low water profile is at first to rather accentuate its irregularities, the considerable slopes at the bars being somewhat increased. But this effect is usually but temporary because the contracted channel naturally deepens through the eroding velocity produced; or is excavated to the desired depth if the bed is too firm to be affected by the greater velocity of the current. The ultimate result is, then, somewhat of an increase of the slight slopes in the pools, and their reduction at the shoal places by an amount which is often considerable. This characteristic of a general tendency toward a greater uniformity of low water slope has been noticed in model experiments and marked in actual experience. Typical instances of the latter are those of the Garonne, where maximum deviations from the average slope of 4 or 5 ft. were reduced after the narrowing to less than one-third as much, and original slopes of 1 or 2 ft. per thousand were diminished proportionately; and the Little River Shoals of the Tennessee, on which the maximum fall of one in a thousand is only about half that which previously existed.

A simultaneous reduction of the variation in depth at low water is characteristic of the effect of works of lateral contraction. This is more largely the result of deepening at the shoals than of the filling of the pools, and therefore it particularly improves the navigability of the river concerned. The amount of such amelioration so obtain-

able may be illustrated by experiences such as those of the Garonne, where minimum depths exceeding 3½ ft. were found over the shoals which had previously limited the navigable depth to 2 ft.; of the Rhone, whose low water navigability was limited by 5 bars in 1878 to a depth of hardly 20 in. but whose training works had everywhere produced at least 4 ft. of water in 1892, and whereas there were originally 111 shoals of a depth less than 5 ft., the result showed only 12 bars of equal limitation; and of many improvements in this country where the navigable depth has been increased from 50 to 100 percent.

The effect of contracting works of ordinary height upon the elevation of the water surface seems to be negligible as far as the results of experiments on models is concerned. Experiences upon rivers themselves indicate that as a general fact the permanent change is usually very slight.[1] Observations for decades upon the Elbe, Memel and Garonne Rivers illustrate the usual results of an immediate elevation of the low water surface resulting from the narrowing of the channel, but followed by a gradual lowering of it as the river adjusts itself to the new conditions of regimen until finally the low water surface is as low or lower in the contracted channel than it was in the natural one. The consideration of the effect upon the high water level is of more consequence. The generally marked raising of the low water surface within the restricted channel as soon as it is narrowed, together with the fact that the original high water cross-section is reduced by the area occupied by the contracting works, has led many to the conclusion that the combined influence must be the elevation of the flood crest and a consequent augmentation of destructive overflows. Such reasoning ignores the later lowering of the low water surface, as just mentioned, as well as the particularly important element of the change in velocity due to the modified slope, cross section and mean depth, a change which need not be large to neutralize the conditions tending to raise the high water plane. Careful observations on the Memel during the years 1875–92, during and shortly after the construction of the regulating works, showed irregularities which characterize the period while a river is adjusting itself to the new conditions; but comparison of the water levels reached by equal volumes of discharge before and after that period show practically no difference for medium and high water stages. Similar investigations on the Vistula, Garonne and other rivers

[1] Brochures 1 and 4 of the Eighth International Congress of Navigation.

justify the general conclusion that the favorable tendencies usually compensate for the adverse conditions as ordinarily encountered, so that the ultimate effect upon the flood level is to leave it practically unchanged.

57. Auxiliary Dredging, and Extensive Excavation.—The period following the completion of works of regulation, during which a river is adjusting itself to the new conditions imposed upon it by works of lateral contraction, is generally of considerable length. During these years the channel is deepening in places and the eroded material is depositing in new locations which not infrequently interfere with the navigability of the stream. This adverse situation is always accentuated if the river carries considerable silt or entrains sediment and detritus. It is consequently a quite general experience that some dredging is necessary to maintain the desired depth in portions of the regulated channel. Of course the amount decreases with the lapse of time, especially as the contracting works are adjusted to prevent the persistence of the shoaling. This appears in the records, for instance, of the Waal in which the maintenance of the channel depth required an average of more than 10,000 cu. yd. per mile of dredging during the decade in which occurred the gradual adjustment of the river to its new regimen, but less than 7000 cu. yd. per mile of channel at the end of that period.

A relatively greater amount of auxiliary dredging is necessary when a considerable increase of navigable depth must be maintained during the construction of the works of improvement. This procedure has been followed on many streams, such as the upper Mississippi in the 658 miles from St. Paul to the mouth of the Missouri. In 1907 the project for the improvement of this portion of the river was radically modified, the new plan involving a minimum depth at low water of 6 ft., or one-third greater than that of the previous plan. This involved a further contraction of the low water channel to a width increasing from 300 ft. at St. Paul to 1400 ft. near the mouth of the Missouri. The construction of these permanent works is supplemented by excavating to the required depth at shoal places which are not yet kept open by regulation. Hydraulic dredging has been found by experience to be the most efficient method of removing the sandy sediment that constitutes the greater part of the material which at present must be periodically excavated. There are eight dredges in commission which in general resemble those described in Chapter VIII, except that they are much smaller, no jet is required for throwing the sandy material into suspension, and the suction

pipe is supported by a catamaran pontoon instead of by the forward part of the dredge itself. This method of mounting, with the flexible joint provided, permits of the constant swinging of the suction pipe through the greater part of a half-circle; and so allows the excavation, at each sweep, of a curved strip of sediment from 1 to 4 ft. wide and perhaps 100 ft. in length. The spoil is carried through the discharge pipe to a place of deposit from which it is unlikely to again be carried into the channel. As much as possible of this dredged material is employed to the advantage of the improvement, in such ways as filling secondary channels behind islands which have slight currents, filling the space between groynes and behind training walls, depositing it on the bed of the river at the location which contracting works are to occupy and thus lessening their cost by reducing their depth, utilizing it for the core or body of traverses when protected by layers of brush and stone, etc. The average amount of dredging during the last three years on the upper Mississippi River, thus necessary as auxiliary to the works of regulation, has been about 2700 cu. yd. per mile of channel; and its field cost has averaged slightly more than 5 cents per cubic yard. The quantity to be removed will naturally become less as the contracting works approach their completion.

Sometimes there are places where the channel of a river is so crooked that its navigation is difficult. An instance of this kind occurred at Buzzard Bar in the Warrior River. Its original condition is indicated by Fig. 57 (p. 224);[1] and the regular channel formed by excavation through the more direct but very narrow and shallow arm of the stream is shown in Fig. 58 (p. 225).[1] This rectification of the channel was made in 1897 at an expense of $4.73 per linear foot, the excavation itself costing an average of 13 cents per cubic yard. Another instance of the same sort on a larger river is that of Coon Slough on the upper Mississippi about 10 miles below La Crosse, Wisconsin. At this place Island No. 120 divided the river into two branches, the wider and more direct one being very shallow at low water while the deeper one was very difficult to navigate because there were two bends quite near each other at its upper end which were very short in radius, each of which involved a change in direction of more than 90 degrees. The considerable expense involved in the rectification of the channel here caused the authorities to postpone its accomplishment in the hope that the currents of the river would gradually accomplish the desired result; but it became

[1] Report, Chief of Engineers, U. S. A., 1897, p. 1684.

worse rather than better, until it constituted the most difficult place to navigate in the upper Mississippi. Consequently it was improved in 1895, by cutting through the upper end of the island and the bars above its head so as to form an easily curving channel from the main river into the Coon Slough branch. The significance of the improvement can best be realized by noting that the radius of curvature of the new channel curves exceeded ½ mile, or about ten times that of the original ones. Groynes were also constructed to guide and hold the current in its new course, closing the shallow arm completely at low water stages. The improved channel is 400 ft. wide and its depth exceeds 6 ft. at low water. It is estimated that this rectification involved the moving of about 1,170,000 cu. yd. of sand, but more than five-sixths of this was accomplished by the erosive action of the current after the cut had been artificially opened enough to draw the water through it. In excavating the new channel about 62,000 cu. yd. of earth were removed by scrapers at an average cost of about 10 cents per cubic yard; and the remainder of the machine work was done by dredges at a unit cost averaging about 14 cents.

The most severe situation involving excavation to secure an improved channel occurs when layers, ledges or masses of rock reach so near the surface as to limit the depth of water. Occasionally the rocky shoals are so extensive and cause such a considerable fall that a lateral canal is the best way of overcoming the difficulty. But ordinarily the remedy is the excavation of the rock to form a channel of the required width and depth throughout the length of the obstruction. The longitudinal plan is often straight in outline, or consists of straight segments joined by small angles or short curves, instead of the curved outline generally advocated in ordinary channels, because the slope is usually so considerable that there is no danger of the deposit of sediment; although there are instances, like that of the Iron Gates of the Danube, in which the form is curved throughout. In case the effect would not seriously increase the high velocity usually occurring in such a situation it is frequently the practice to place the excavated rock in such position in the river bed adjacent to the cut that a greater volume of water will be carried into the excavated channel than would otherwise reach it. Excavation through rock has been necessary at the Rock Island rapids and at other shoals of the upper Mississippi; on the upper Hudson River; on the German Rhine where, in fact, about two-thirds of the total cost of improvement has been expended in rock excavation in its

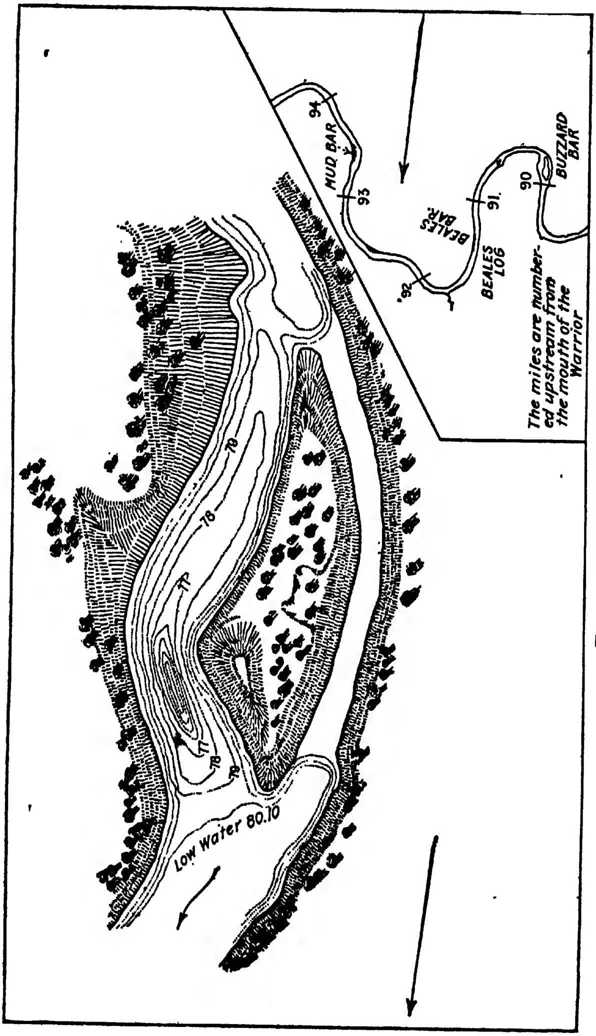


FIG. 57.—A difficult passage.

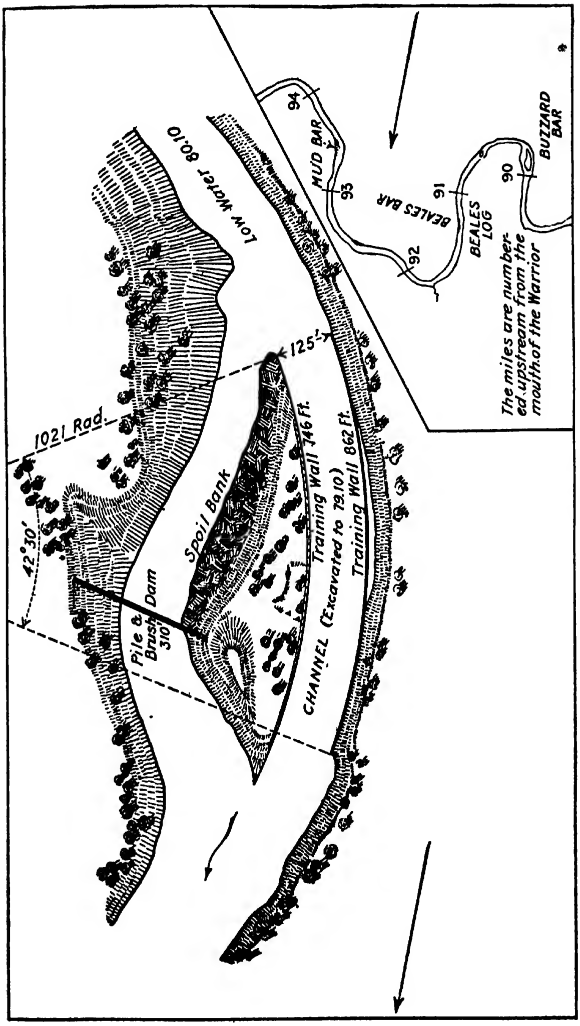


FIG. 58.—The rectified channel.

middle portion; in short, nearly all rivers, whose improvement for navigation has been undertaken, have presented more or less extensive shoals whose deepening has necessitated the removal of rock-barriers and obstructions.

One of the most recent operations of this kind has been in progress at Tuscumbia Bar of the Tennessee River for several years. The project provides for a channel 150 ft. wide and 5 ft. deep at extreme low water through this shoal about 2 miles in length whose depths were frequently less than 2 ft. The method used was the usual one of drilling, blasting, and excavating the shattered stone by dredging. Each of the three rafts was formed by rigidly connecting together thirteen small flat-boats by a framework of timber on which eight steam-power drills were mounted, intervals of 1 ft. being left between boats through which to operate. Pipes were employed to prevent the drill holes from filling with sediment and to facilitate the placing of the explosive charge. At each successive position of a raft there were seventy-two holes drilled, the charge of each being from six to ten sticks of 80 percent gelatine dynamite, and all were fired simultaneously by electricity. During the year 1911 the holes were drilled at intervals of 7 ft. in one direction and 6½ ft. in the other, and were carried to a depth of 2 ft. below the level of the finished bottom. The dredging of the broken rock during the following winter revealed the fact that there were very numerous places between drill holes where the solid rock still extended above the proposed bottom level; consequently the spacing of the holes was reduced to 4 by 6 ft. and they were drilled to a depth of 9 ft. below the low water surface. The fact that the rock was an unusually hard, flinty limestone not only caused this tripling of the drilling required to secure the adequate shattering of the rock; but it also necessitated the frequent sharpening and retempering of the drills, the edges often becoming blunt after advancing only a few inches; and of course it greatly increased the cost. The pertinence of the last statement is indicated by the fact that the expense per linear foot for drilling in the comparatively soft oölitic limestone of Buck Island Shoals, 3 miles below, was little more than half that at Tuscumbia Bar where the field cost averaged $0.46 for the 28,831 linear ft. drilled in 1911, and $0.483 per linear ft. for the total of 85,708 ft. drilled during the season of 1912. Each year nearly a half mile of channel was covered. In the year 1911 the total volume of rock, measured in place, which was broken by the blasting sufficiently for removal by the dipper

and orange peel dredges amounted to 23,155 cu. yd.; the average cost of this drilling and blasting, including field expenses, deterioration of plant and overhead charges, was $0.76 per cubic yard or $0.617 per linear foot of drill hole. During 1912 there were 76,185 cu. yd. of rock loosened at an average total expense of $0.677 per cubic yard, or $0.595 per linear foot drilled. The amount of dynamite required for shattering the flinty limestone averaged three-fourths of a pound per cubic yard, and each cubic yard of rock which was loosened required an average length of drill hole of 14 in.[1]

Rock excavation of greater magnitude has been carried on at the Rock Island Rapids of the upper Mississippi River during the latter third of the last century and the first years of the present one. The plan involved the removal of the hard limestone rock occurring in detached reefs and of occasional great granite boulders at twelve different localities, to open a channel 6 ft. deep and 400 ft. wide. Nearly 80,000 cu. yd. has been removed by blasting and dredging under water, the total cost of which has averaged $4.14 per cubic yard, and the mean amount of dynamite required was about 2.3 lb. per cubic yard excavated. There was a total volume nearly three-fourths as great which was removed, in the earlier years of operations, by drilling, blasting and excavating in the open after the water had been excluded from the sites by cofferdams; the cost of this averaged $11.35 per cubic yard.

58. Conditions that Require the Limitation of Depth.—Not only is lateral contraction quite effective in improving rivers, but an artificial limitation in depth is often found also necessary. It might seem that there can be no objection to depths, greater than that required, existing anywhere in the river; and as far as that fact is directly concerned excess of depth is not objectionable, but rather the reverse. Yet experience has frequently shown difficulties resulting indirectly from the existence of depths greater than that of the project, especially for one which requires about all the low water discharge; and many such defects have been most economically and effectively removed by correcting them through the construction of sills, which are called in writings on the subject by various other names, such as sill dams, ground dams, ground weirs, ground sills, bottom sills, lower groynes, submerged groynes, submerged dikes, cross dikes, cross weirs, sloping sills, bed sills, bottom ridges, submerged spurs, etc.

Occasionally conditions are found where a channel is so deep that

[1] "Professional Memoirs," Vol. 5, pp. 545-558.

its navigable width is deficient. When this occurs the method usually found most effective in widening such a place is the reverse of that already discussed at length as ordinarily used to produce a deepening of shoal places; that is, in this case to substitute in the hydraulic formula $b\sqrt{d^3} = \frac{Q}{c\sqrt{s}}$ the required value of "b" and solve for "d." This value, conservatively modified as observation and experience indicate, gives the depth at which the crest of the sill is to be placed in order to induce the desired widening of the channel. This procedure has been satisfactorily applied on the Weser and other continental rivers. It seems quite possible to produce the desired widening otherwise; that is, by flattening the usual steep underwater slope of the concave bank to perhaps 1 on 3 as discussed in Chapter X.

A frequent use of sills is in contracted channels which (because their longitudinal curvature is too slight to properly correlate with the velocity in a way to produce axial flow, or from other defects of design) fail to secure the expected navigability because of the lack of longitudinal continuity of deep water. In such cases, while there is often a sufficient depth and width, it extends laterally (even sometimes occurring at the convex bank) in such a tortuous and shifting way that the coexisting moderate shoal places greatly hamper navigation. A usual remedy for this condition is the construction of sills in order to equalize the depths and straighten the channel, as on the Waal (in connection with further contraction as indicated in Figs. 30 and 31) and the Rhone, on which the use of sills achieved the final success of regulation, which had been but partial previous to their use.

While it is often possible to assist or even to secure the desired effects discussed in the two last preceding paragraphs by means of further contraction or by correcting the alignment and curvature of the lateral works of regulation, the former is often less effective and the latter method would frequently prove to be much more expensive.

However, sills form the only way by which the elevation and slope of the water surface can be effectively controlled. It often occurs that the velocity existing in a contracted channel is an eroding one; and if the consequent lowering of the bed proceeds to such an extent as to produce undesirable results, such as the reduction in elevation of the pool above so much that shoals develop in it, the level of the

water surface can be held at an elevation to prevent this eventuality by the construction of sills to check the erosion of the bed. This procedure has formed an essential part of the regulation of the Weser, and has been much used on many other rivers.

In situations in which it is desirable to reduce the velocity of flow in the river channel in the interest of its navigability, the slope must be lessened either by decreasing the difference of elevation between the water surfaces of the pool above and of that below (by lowering the former by permitting the erosion of the bottom of the connecting channel or by excavating it, or perhaps by raising the latter by using sills beyond it) or by lengthening the distance in which the fall occurs (possibly obtained by allowing erosion of the contracted channel at its upper end, but surely secured by advancing its lower end). The latter method is the one usually adopted because the effect is thus entirely localized, and so is not detrimental to neighboring portions of the river. It consists in continuing the contracting works through the necessary additional distance down-stream, and the construction of sills throughout the length of this channel at the same slope of crests as that planned for the water surface. Sills have thus been extensively employed, as on the Rhone and Waal, in special cases on the upper Mississippi, and on the Weser where "most signal proof has been given that a raising of the water level with an excellent equalization of the falls can be attained by the application of ground dams."[1]

It is rare that the erosion of the bed of a contracted channel is itself an evil. Surely the added amount of sediment, compared with the quantity already burdening a sedimentary stream, is comparatively small. The river Weser seems to furnish an instance where it was, as stated in the paper last quoted, deemed inadvisable to permit further bottom erosion in order to prevent an increase in "the great difference between low water and the surface of the country already to a large extent prevailing." Another example of a different kind, in which continued erosion would have produced serious results, is furnished by the need for control of the place where the Red River flows into the Mississippi, which is coincident with the head of the Atchafalaya River. It was necessary to prevent the growing tendency of the Atchafalaya to divert too much of the flow of the Mississippi, to keep unobstructed the high water relief which the Atchafalaya offered against excessive floods on the Mississippi River below

[1] Paper 2 of the Twelfth International Congress of Navigation.

this point, and to preserve the freedom of navigation into each river from the other two. Therefore mattress sills, loaded with stone, placed on the deepening bottom near the head of the Atchafalaya, have controlled the situation with entire success.

A particular adaptation of method to produce desired effects is furnished by some French and German experiences in rectifying channels and securing quite symmetrical improved sections by a skillful coördination of the details of design of both the works of lateral contraction and of the sills in such sequence of form and position that a more definite control of the current is obtained. On the Rhone, for example, in thorough correlation with the more usual works of contraction, a particularly effective agency of improvement consisted of sloping sills, or submerged spurs (*épis noyés*), whose efficacy in forming a regular and relatively permanent channel proved successful where all other known means had previously failed.

59. The Design and Construction of Sills.—The materials used in the construction of sills are generally the same as those employed in groynes or training walls in the same locality, as already described. Both contracting works and sills are planned to articulate advantageously; and, as mentioned in the last paragraph, a thorough correlation of their effects is most important.

Sills usually extend entirely across the channel, normal to its axis; although, as in their use in the Waal where they extend from the shallow edge to about one-third of the way across, it is sometimes unnecessary to incur the expense of constructing them throughout the entire width. They are also built with either a level top, or, more advantageously for the influence obtained in directing the current toward the center of the channel, with their crests inclining downward from the banks. The top of a sill is usually somewhat below the elevation defining the channel depth as projected, although it is sometimes placed at that depth, as in the Waal. In the Weser the height of the crests of the sills at their outer ends was fixed at 1 ft. below the normal bed, and they sloped toward the center of the channel at a grade of 1 on 40.

Details of construction, such as their sectional dimensions, vary with the velocity of flow and other agencies which must be successfully resisted, and with the materials and character of the sills. On the Waal the width of crown was about 17 ft. for that portion constructed wholly of gravel, and for that part consisting of a core of sand protected by fascines covered with gravel, the top width was fully 40 ft. and the side slopes were 1 on 1½; in deep water gravel is

used for the base of these sills as a fill from the bottom up to an elevation of 15 ft. below low water.

The distance apart at which sills should be located depends partly on the purpose to be served, but largely on the regimen of the river at the place. It has been a frequent experience to find that the expected results are not attained because of too great a spacing; as on the Waal, where the especial reason for their construction was the equalizing of depths and the rectification of the navigable channel, the sills were at first built 656 ft. apart, but the effect was not satisfactory until the spacing was reduced to half that distance. When sills were first constructed on the Weser to reduce the slope of water surface, they were placed so far apart that the results were very disappointing and it was doubted if they could be made effective for this purpose; but now that the spacing has been made only 41 ft. the result is entirely satisfactory. The model experiments made at Berlin in 1907 indicated the same general facts; when the spacing was made not more than one-third to one-fifth the width of channel there were marked effects in the widening of a deep and narrow stretch or in raising the water surface in a shallow one, but these effects were lessened with increasing rapidity as the distance between sills was lengthened above the ratio just mentioned. Undoubtedly the close spacing necessary to produce the desired results was mainly due to the considerable slope employed in the experiments, which was about 1:650. No advantage resulted from the filling of the intervals between sills to the level of their crests.

Of course sills produce a concentration of slope of water surface, due to their submerged weir effect; but this exists for so short a distance that, even when the descent is unusually large, it does not seriously affect navigation. Experience on the Ohio River is said to indicate that there is no especial difficulty occasioned by a sudden drop of water surface as great as 1 ft.

An instance of a very moderate cost of sills is that of the type mentioned as employed on the Waal, which was $5.20 per foot of length; and the opposite extreme is illustrated by the estimate of $60.00 per linear foot for the massive construction of piles, timber, stone and concrete considered for the great sills planned in connection with the proposal to increase the navigable depth of the Mississippi River to 14 ft.

CHAPTER VII

THE PROTECTION OF ERODIBLE BANKS

60. The Purposes Served by Bank Protection.—The term bank protection, in its general sense, refers to works designed to defend, against the river current's attacks, the integrity of all that rather steeply sloping margin extending from the nearly level portion of the bed of the river to its high water mark, or to the top of the immediate banks if they are below the high water line. This includes, then, both that part of the bank which is above the ordinary river stages, whose protection is often necessary because of the violence of the surging rush of the flood waters, and also that sloping part below the water's edge, whenever there exists any lateral erosive action of the current which not only disintegrates the soil of the side of the channel but also progressively destroys all the exposed bank above through its consequent caving or sloughing into the stream. In general, wherever the velocity of flow is an erosive one for the material composing the bank, it requires protection from this disintegrating action; although circumstances may sometimes mitigate the rigidity of this rule with regard to that portion usually above water, and so infrequently exposed, when it is defended against being undermined. Such a plan is followed on some Russian rivers, where it is considered necessary to only protect the bank below the low water surface, leaving the exposed bank to be gradually eroded until it attains a stable surface slope of its own; except in situations where there is some especial need of complete protection, such as adjacent towns or quite valuable lands.

There are two particular reasons for protecting erodible river banks in the interest of its navigation; the holding of the stream to a permanent channel and the avoidance of a great part of the silt and detritus whose recurring deposit at various places greatly complicates the work of river improvement. With reference to the latter, it is only necessary to recall that the computed annual erosion of the banks of the Mississippi River between St. Louis and Donaldsonville is about 940,000,000 cu. yd., while the contributions of its principal branches are given as about 400,000,000 for the Missouri,

36,000,000 for the Ohio, 5,000,000 for the Arkansas and 6,000,000 for the Red, to realize the particular need of preventing the erosion of the river banks in order to decidedly ameliorate its natural condition.

The fixation of situation of the channel is directly necessary to preserve the effective action of works of regulation in the position in which they are built; as, for example, when the improvement takes the form of lateral contraction entirely on one side of the stream the resulting velocity of the current in the restricted channel is liable to be an eroding one; and if the bank is left unprotected its resulting recession will frustrate the object intended. The permanent establishment of the channel is also essential, indirectly, because a change in its position involves variation in the curvature and other characteristics of its banks and bed which, though occasionally not important at the immediate locality, so alter the position, direction and strength of currents below or above that the results there may be very serious. It is a common experience on rivers with unstable beds to find a place, whose regimen may have been fairly definite for years, that displays a decided and even violent change of character when the progressive influence of altering conditions occurring at another point has reached the place in question; as when a bank which has been comparatively stable is found to be caving. Numerous cases have occurred in which works of river improvement have been adequate until a change in characteristics of flow at the point makes them ineffective or even destroys them. The essential remedy for such unfortunate results is the stabilization of the river by definitely protecting all its erodible banks. Often this purpose is especially significant; although the defense of cities and other valuable property, the preservation of access by boat to the more or less expensive and necessary transfer facilities of landings and terminals, the safeguarding of levees against destruction by undermining, the prevention of such radical disturbances as result from cut-offs, the reduction in silt and sediment, and the preservation of channel dimensions as just mentioned are all considerations of great importance also, any one of which may be the paramount one at a particular place considered.

61. The Use of Groynes, Training Walls and Bank Heads.—Groynes and training walls, as ordinarily employed in river regulation, do serve to protect the banks, in front of which they are built, up to the level of their crests when they are properly spaced and constructed with their accessory defenses against undermining and

a flanking scour. Yet inasmuch as the safety and convenience of navigation, as well as the efficiency and economy of the works of regulation, generally favor the location of the improved channel close to the concave bank, which is the eroding one, it is usually found desirable to protect the latter by a method specifically planned to secure that result. The trend of practice has for some time been in this direction; as on the Weser where "groynes in concave shores are to be replaced by bank defense works." Of course such structures for contracting the channel at crossings will continue to serve as protection for the lower banks opposite them; and there are especial cases, such as a harbor front, where conditions may be such that facilities for loading and unloading of boats will make other measures preferable. But usually it is advisable to adopt a method which will directly accomplish the purpose, particularly as eroding banks usually occur in the deepest and most persistently perverse parts of the river.

There are two general classes of improvements constructed expressly for bank protection; one in which comparatively massive structures are built at intervals, and the other forming a continuous covering. The former are intended to act by diverting the strength of the current away from the bank, and thus to create between them a region of comparatively quiet water lacking an erosive velocity; and the latter form a resistant covering of the soil of the whole length of the bank, thus creating an armor against which the velocities of the current are powerless. Continuous protection is known by the general name of "Revetment"; while that involving the construction of individual units at intervals has been of two principal types, "Bank heads" and "Spur dikes."

In the years 1897 and 1898 eleven bank heads were constructed in some of the concave bends of the lower part of the Missouri. They were built of riprap placed in curved outline with the faces forming elements of concentric conical surfaces, as indicated in Fig. 59.[1] Each structure extended through about 100 degrees of an arc whose radius was perhaps 350 ft., and its middle portion projected slightly into the river. After shaping the necessary excavation the rock was placed in a layer averaging about 5 ft. thick, the curved level top being perhaps 20 ft. wide and reaching a few feet above the low water plane, and the outer face inclined downward from this level at the slope of repose of the stone; while

[1] Report, Chief of Engineers, U. S. A., 1898, p. 3548.

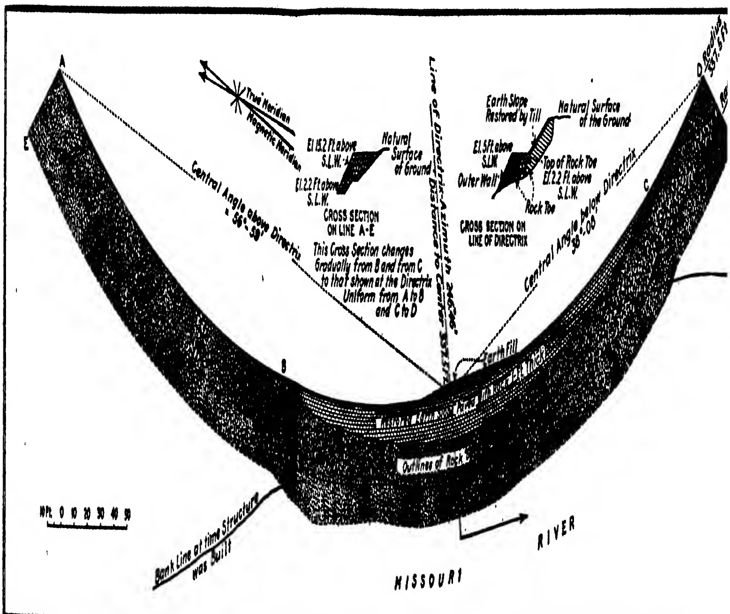


FIG. 59.—Plan of a bank head.

(Facing page 234.)

above the berm a protecting paving of rock, at a slope of 1 on 2, extended to the top of the river bank. The middle and up-stream portion was made particularly massive, requiring about 125 cu. ft. of stone per linear foot; but this quantity was gradually reduced in the down-stream arm until only one-tenth as much rock was placed at its extreme end. The appearance of a completed structure is shown in Fig. 60.[1]

It was the expectation that each bank head would serve as a point of control for about a half mile of river. If this hope were realized,

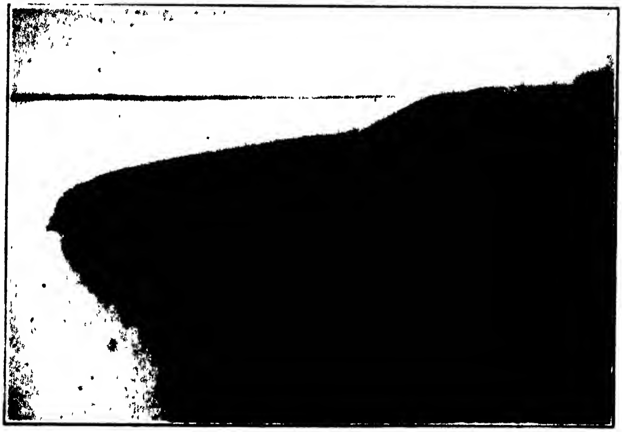


FIG. 60.—View of a bank head.

it would mean an expenditure of less than half that required for revetment, as the first cost of the bank heads averaged hardly $8000 each. However, the experience of the next three years proved that the structures as built were very vulnerable, the swirling waters attacking the exposed banks above and below them and also the unprotected river margin beneath them. The effect which the placing of strong defenses at intervals has upon the activities of river currents is indicated by the instance in which the river bed just in front of one of the bank heads was scoured to a depth of 35 ft. below low water, increasing to 64 ft. a short distance out. Reconstruction, repairs, extensions and new work, and especially the heavy defenses of piles, timber, brush and stone found necessary to save the bank

[1] Report, Chief of Engineers, U. S. A., 1899, p. 3744.

heads from undermining, were necessarily so extensive that the supplementary expenditures had exceeded the original cost by nearly 40 percent in that short time.

The purpose for which the bank heads were built was to secure a directive influence upon the current and to protect the caving banks, at a less cost than is involved in a continuous revetment. Their anticipated efficacy in producing a regularizing and guiding influence upon the current did not prove to be justified; changes in the condition and regularity of the main channel continued to be, in general, as severe and fully as eccentric as before their construction. With regard to the protection of the banks it was expected that caving would continue between bank heads, but to a lessening extent until it would cease when the resistances to flow along the eroded concavity became greater than on a more direct line farther out. But the severity of the attack of the river upon the banks was so considerable that in one place the recession exceeded 500 ft. in two years, and was nearly as serious in other cases; so that auxiliary defenses had to be constructed to save the river banks from excessive erosion, as well as to guard the bank heads themselves from being flanked and destroyed by the currents, as already stated. Under the revised project of 1912 the bank protection consists of revetment.

62. The Design and Construction of Spur Dikes.—Spur dikes (occasionally called buttresses or spurs) are similar in principle to the sloping sills, mentioned a few pages back, in that both make use of the direct principle of deflecting the current away from the bank; although the effect sought is different, being in the case of the latter to secure a better channel, while, in the use of spur dikes, it is primarily to prevent bank erosion. The protection afforded is effected particularly by their height and spacing, rather than by their length which is determined by the sloping distance to be protected. The direction of their axis is also important, because it seems to have considerable effect upon the degree of deadening produced on the current between them; and it becomes a very complicated and uncertain proposition for spur dikes extending above low water when the current at the bank in question varies in its line of flow at different stages. The position of these comparatively low structures is ordinarily planned to be approximately normal to the bank, but some authorities believe that they should point somewhat downstream in order to minimize the troublesome tendency to eddy action between them, which is so likely to occur that many consider it inevitable, with serious results.

Spur dikes, like all other regulating works, must have an adequate protective foundation when on erodible soil, which generally consists of a well-built brush or fascine mattress extending throughout their length and projecting sufficiently beyond their ends and sides to adequately guard against undermining. On the lower Mississippi River, for example, the foot mattresses were generally made of sufficient length to stretch from the low water margin outward to the deepest water. Their width was 200 ft. in most cases, of which perhaps two-thirds extended down-stream from the axis of the superimposed cribs in order to the better defend the more exposed lower edge against scour. In some cases the width of mattresses was reduced to 120 ft. in order to lessen the cost; but a considerable erosion at the lower edge gave indication that the narrowing had been carried too far to suit the conditions existing at the places in question. The mattresses were from 18 to 30 in. thick, and from 7 to 14 lbs. of rock per square foot of surface was used in sinking them. The different kinds of mattresses are described later in this chapter.

Spur dikes for bank protection consist occasionally of sloping mounds of stone, rubble or concrete. Sometimes they are of cellular construction of brush in which the open part is expected to fill with alluvium, as in the David Neale Dike System described in Vol. 12 of Proceedings of the American Railway Engineering Association; or where the cells are formed of galvanized wire frames filled with gravel or stone, as on the French river Drac. Generally spur dikes are built of timber cribs, but varying in detail from a simple framework of wood which gives the needed strength to hold the filling of brush and stone, to a more solid crib construction filled entirely with stone. Sometimes they have been built to extend considerably above the low water surface, as well as below; but the more usual and better practice is to limit their highest part to this level in order to avoid the decay of the wood, and to protect the exposed bank above by a stone revetment.

During the last seventeen years of the last century scores of spur dikes were constructed on the lower Mississippi River to provide protection for such localities as the harbors of New Orleans, Natchez, Greenville, Helena and Memphis. They were all in the deep water characteristic of the concave curves of that great stream, the soil being alluvium and therefore easily erodible. A typical view[1] of one

[1] This and the following figure are photographs of models in the Civil Engineering Museum of Washington University.

of these structures is shown in Fig. 61 which represents to scale the largest one built. The mattress is seen extending from the margin of the river, at the top, downward for a distance exceeding 500 ft. to the lowest part of the bed at a depth of more than 150 ft. The characteristic very steep slope of the underwater bank is noticed near the upper portion of the view; and this not only caused some apprehension concerning the possibility of future slides at localities where it sometimes was found to have slopes as great as two on three or steeper, but it also required a thorough hydrographic survey to

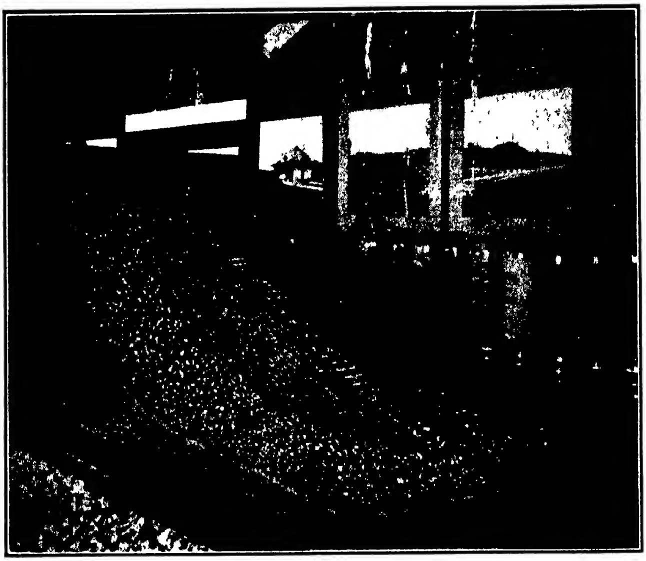


FIG. 61.—A complete spur dike.

enable the planning and sinking of successive cribs in such a way that the top one should extend continuously downward from the edge of the river at an inclination throughout of about 1 on 3. Seven tiers of cribs were required for this spur dike, the lowest one being quite short and having a width of about 60 ft., while those placed later were successively longer and narrower, the upper one having a length of 450 ft. and a width of 16 ft.

The details of the cribs are better shown in Fig. 62 where the mattress with its weight of stone is seen at the lower edge at the right, and above it appear three tiers of cribs each of which is about 6 ft. nar-

rower than the one immediately below. The skeleton of each crib consisted of frames of sawn timber placed vertically as appears in this figure, one at each side and intermediate ones in number depending upon the width of the crib, their usual spacing being 8 ft. but sometimes more. The upper and lower longitudinal stringers of each frame were rigidly connected by stanchions at intervals of about 5 ft. except at the ends where two were placed close together for greater strength. Several layers of poles, whose length equaled the width of a crib, were fastened to the upper side of the lower

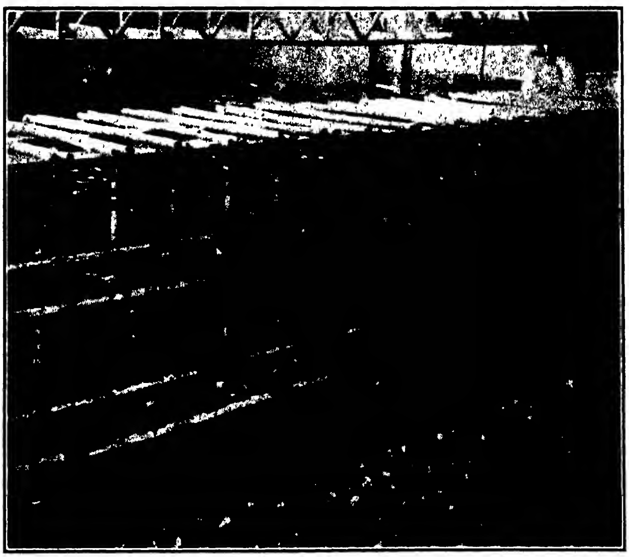


FIG. 62.—View of details of a spur dike.

stringers, and crossing them were other layers of poles placed parallel to the longitudinal axis of the crib, and likewise extending throughout its length and width; the thickness of each layer was about 16 in. The poles of both these bottom layers were thoroughly wired and spiked to each other and to the timbers of the frames. In the remaining space poles were laid in single layers alternately in transverse and longitudinal directions, reaching to the top of the crib; but in such a way that, in every second section, open spaces were left of a form of inverted frustums of pyramids for the purpose of containing

the stone necessary to sink and hold them in their final position. Two transverse poles were also laid across the crib to connect the top stringers at every stanchion as shown in the last figure; and these, like all the others, were thoroughly fastened and bound together and to the sawn timbers by spikes, wires and small cables, so as to form the whole into so strong a structure that it would successfully resist the severe stresses occurring in its sinking. Treenails were also employed wherever possible, especially in the joints of the frames, to maintain the integrity of the cribs in position after the metal should be destroyed by corrosion.

The cribs were framed on barges, and when partly completed they were launched into the water above the position which they were to occupy and held by lines extending to the mooring barges alongside. In this position they were finished. Cables were then passed from the crib to adequate anchorage at the bank to hold it from sliding outward, the numerous supporting cables were finally adjusted and the location of the crib was made exact, and then it was uniformly lowered by the slip lines to its intended position after the compartments were filled with rock the weight of which amounted to 6 or 8 lb. per cubic foot of volume of the crib.

The distance apart at which spur dikes are built is a most vital consideration in its effect upon their efficiency, and the desire to keep the expense at a low figure has often led to so great a spacing that the eddying waters between them have continued the erosion which they were intended to prevent. The distance between them must necessarily depend upon the character of the river and that of the material composing its banks. On the Mississippi the spacing of the spur dikes of the Citizens Bluff Protection at Memphis was about 500 ft.; at Greenville it was about the same; while at New Orleans it has averaged somewhat more. At Helena the original spacing was 400 ft. Above Natchez they were built at a distance of 450 ft. apart; but this proved to have been too great, as the deepening of the bed between them in a year's time was generally about 30 ft. and the recession of the banks was from 25 to 100 ft., necessitating extensive subsidiary works of bank protection.

The first cost of such comparatively temporary works as the Neale system has been from about $3 to $8 per lineal foot of bank, depending on the severity of the exposure and the conditions of construction. That of the much more permanent spur dikes and upper bank protection at Memphis and Greenville was about $20 per foot. The

cost at New Orleans was somewhat more, exclusive of capital charges on plant employed in the work. For those at Helena it was nearly double the amount just mentioned, largely because of local exigencies and losses during construction. While the lighter units placed above Natchez cost only about $9 per foot of bank originally, the additional expenditure for repairs and intermediate revetment required to preserve the bank from further erosion, during the next two years, had brought the total to an average of $13 per foot of bank. The cost of individual spur dikes ranged from less than $5000 to more than $25,000, but averaged $10,000 or $12,000 each. A more definite measure of expense is a unit one; so considered it may be stated that the field cost of the foundation mattresses usually ranged between 5 and 8 cents per square foot, and that of the cribs between 3 and 5 cents per cubic foot.

63. Results of Experience with Spur Dikes.—Experience with spur dike protection has developed many defects. In cases where timber and brush have been used above the low water surface, its rapid decay has made the work short-lived and has required renewal, as might have been anticipated. The lighter and less expensive types of construction have been found to require a very considerable attention to keep them in repair, thus involving a relatively high annual expense for maintenance. As a general rule their efficiency in the attempt to secure a non-erosive velocity along the bank has been only partially successful, owing particularly to the usual seriously eddying currents produced between them. At Helena the slowly progressive recession of the bank was not only marked between the spur dikes, but at their shore ends the caving had occurred to such an extent that they no longer reached the bank even at low water. Consequently in 1900 mattresses were placed in all the intervals except one, as well as a continuous revetment for a half-mile below them, in extension of the standard revetment already in place above them.

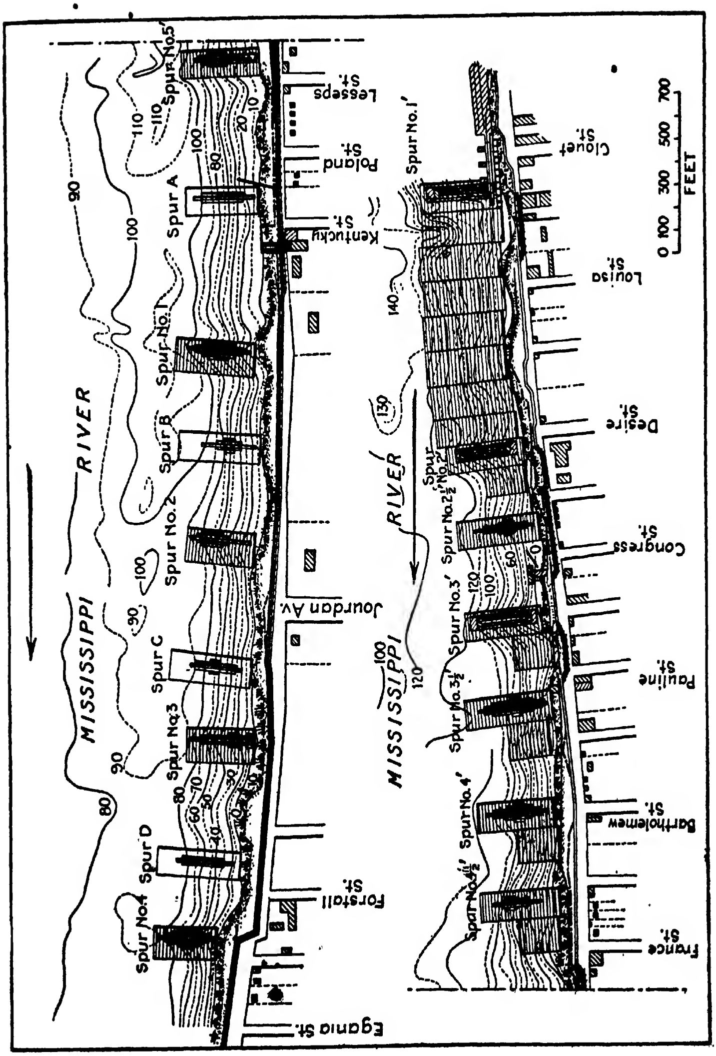
The experiences above Natchez were similar, the recession of the shore proceeding until nearly all of the spur dikes projected outward from salient points of the bank; and at some of them the river was, two years after their construction, seriously menacing their connection with the concave margin. During the next few years repairs were made and their shore ends were extended as necessary. Mattresses were also placed between them to cover all the unprotected bank until, in 1904, there was a continuous defense in place extending from above the upper to below the lower spur dike built six years

before. The construction of twenty-two additional spur dikes here, authorized in 1898, seems to have been abandoned.

The most serious experience occurred in the harbor of Greenville where ten were constructed in 1887. The configuration of the river was such that very strong currents existed throughout the extent of the improvement and the modifying influence of the continuous erosion of the concave bank, on the opposite shore just above, was throwing the point of maximum attack upon these spur dikes progressively further up-stream until, in 1889, the bank above the series was caving so rapidly that there was serious danger of the river flanking them at the upper end, and so destroying them. To prevent this two additional spur dikes were built about one-third of a mile up-stream, but these were destroyed in 1891. The eddying waters between the original ten were producing a generally increasing scour between them which not only caused them to settle but also threatened the destruction of several, notwithstanding the efforts made to hold their connection with the bank by repairs, extensions and the placing of subsidiary mattresses whenever appropriations permitted this to be done. The erosion further up-stream continued to such an extent that, in 1894, the shore line had receded about 1000 ft. and only the lower seven spur dikes remained in position. During the next ten years this concave bank was covered with a continuous revetment which has been effective in holding the river from further encroachment.

The situation at Memphis was quite different from that at Greenville; for the concave bank of the river, opposite and above, has been so definitely held by revetment that stability of current conditions at the spur dikes has been secured. Although their outer ends have settled about 4 ft. the effectiveness of this improvement has continued for a quarter of a century.

The experiences at New Orleans have also been rather favorable. About half the spur dikes constructed there have, with a reasonable amount of attention, proved effective for their purpose without the employment of excessive accessory construction. Fig. 63 indicates the general situation in the least curving bend in this vicinity as existing a few years after its bank protection was begun. In the case of the others, situated generally in sharper bends of the river, it has been found advisable to gradually reinforce the defense by intermediate mattresses until the intervening spaces, as well as the banks above and below, are now covered continuously. The two great structures built in the sharp Greenville Bend at New Orleans in 1889


MISSISSIPPI RIVER
Spur No.5
Spur A
Spur No.1
Spur B
Spur No.2
Spur C
Spur No.3
Spur D
Spur No.4
Lesseps St.
Poland St.
Kentucky St.
Jourdan Av.
Forstall St.
Egania St.
MISSISSIPPI RIVER
Spur No.1'
Spur No.2'
Spur No.2½'
Spur No.3'
Spur No.3½'
Spur No.4'
Spur No.4½'
Clouet St.
Louisa St.
Desire St.
Congress St.
Pauline St.
Bartholemew St.
France St.
0 100 300 500 700
FEET

did not prevent a gradual recession of the bank, which finally became so serious that they appear to have been entirely replaced by a continuous revetment.

64. Advantages, and Materials Used in Continuous Revetment.—The general use of mattresses to check the erosion of the intermediate slopes left unprotected by the spur dikes, and also the entire substitution of a continuous revetment in places where they have failed, are significant results of the attempt to attain the desired effects at a reduced expenditure. Experience with spur dikes indicates that their success is more or less uncertain, especially on banks of easily eroded material and in rapid currents, and particularly at times of higher stages of the river when the currents have a much greater velocity and often are directed against the works at different angles which vary with the height of the stream, thus being very liable to produce eddies and swirls of a destructive character. The general situation has been thus summarized by a special board of engineers: "The only known remedy for eroding banks is their protection both below and above low water by some form of continuous revetment. The experience of the government engineers during many past years has shown that protection of banks by mattress below low water and by paving above low water can be secured anywhere along the Missouri and the Mississippi Rivers."[1]

Revetment has the advantage of definite success in preventing erosion, and in its applicability to a river of any size or condition of regimen. Its particular disadvantage is its cost, which has led to various attempts to modify the plan of operations; such as interrupting its continuity as tried at Fletcher's Bend in the Plum Point Reach of the Mississippi River where the intervening spaces were about 500 ft. long, but without definite success because there was a very considerable erosion and caving of those sections of the bank which were left without revetment.

At the present time it is generally considered essential that the protection should be continuous along an erodible bank. Its termination at the down-stream end may ordinarily be safely fixed at the place where erosion ceases, because the fixation of conditions along that bank will usually prevent any extension of the range of erosive action. Whether or not the up-stream end should similarly coincide with the limit of caving as it occurs at the time is an open question. Experience reveals no uniformity in its indications. In some instances the upper end of the bank defense so built has soon been

[1] H. R. Doc. No. 50, 61st Congress, 1st Session, p. 48.

buried by an alluvial deposit, proving that its construction was an unnecessary expense. In other cases the unprotected bank above has begun to erode and the attack upon it has increased in severity until the upper end of the revetment has been flanked and so destroyed. Actually the decision of this question must depend upon the channel conditions above. If they are stable, the upper end of the bank protection should be coextensive with the erosive action; but if unstable, its extent must be determined by a very thorough examination of the character of the modifications occurring in the river channel above, as well as of the rate at which it is changing. It is because such changes up-stream may make unnecessary or on the contrary may sometimes destroy all kinds of regulation works, as well as for the purpose of decreasing the amount of bar-forming sediment entrained, that it is always desirable to begin the improvement at the upper end of a part of a river, and work down-stream.

There are some kinds of revetment, such as a carefully laid packing of stone, or the patented "Villa System," the "System Adouin," etc., whose materials do not decay when exposed to the air at the lower stages of the river, which may be used both above and below water; but the difficulties and expense involved in their use have rendered their employment quite rare. This question of durability ordinarily leads to a decided difference in materials and construction between those revetted portions below and those above the water surface, mattresses being typically employed for that part which will be always submerged and a stone riprapping or paving usually being constructed on the sloping bank above water. Before grading the bank above water to receive its stone revetment, it is often advisable to sink the mattresses into place upon the sloping bed which has been prepared, if necessary, by removing all snags or other obstructions; or in extreme cases by dredging if the under-water slope is so steep as to be too unstable. This sequence of operations not only avoids the deposit of the earth, if removed from the slope by the hydraulic method, in irregular mounds near the edge of the river bed where it would interfere with the proper placing of the mattresses, but it even furnishes protection to them by settling into their open spaces and so consolidating them, at least to some extent.

65. The Construction and Placing of Mattresses.—The construction of mattresses has had its greatest development, in size, strength and types employed, upon the lower Mississippi River. The kind which was first used at all extensively, a quarter of a century ago, was of the woven type. Such mattresses were built directly above

the position they were to occupy and then were sunk into place. The general operation of construction is summarized from various descriptions, such as that of the Report of the Chief of Engineers, U. S. A., 1891, pp. 3601–6. For the making of the earlier mattresses, of a width not exceeding about 150 ft., two barges were employed which were about 25 ft. wide and perhaps 20 ft. longer than the width of mattress required. They were placed alongside each other with one end at the bank, their sides consequently being normal to the

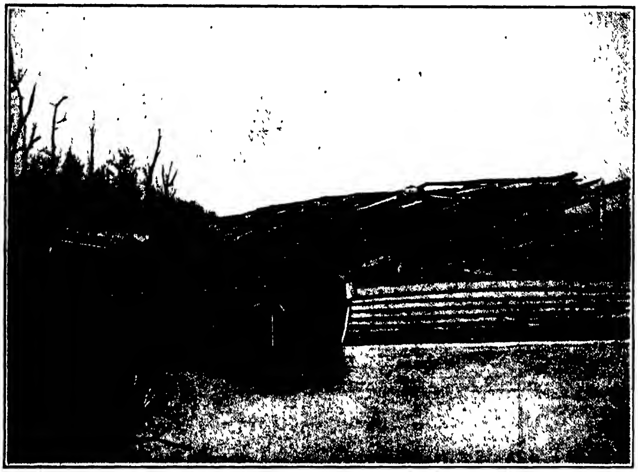


FIG. 64.—Portion of a mattress ready for launching.

direction of flow. The mooring barge was held in this transverse position by cables extending to shore or to anchor piles above; while at its down-stream side the building barge was kept in position by lines extending to the mooring barge. Upon the building barge there were constructed the sloping ways, usually from 6 to 8 ft. apart for the full width of the mattress, inclining downward at an angle of 10 or 12 degrees until they reached close to the water surface in the space between the two barges. The mattress was constructed upon these ways. A general view of a building barge with a mattress covering the ways is shown in Fig. 64.[1] About 1890, when mattresses were often required to have a width of 300 ft., it became necessary

[1] This and the following figure are taken from Report, Chief of Engineers, U. S. A., 1890, p. 3210.

to use two barges of each kind, lashed end to end, to provide for their construction. In building the mattress upon the sloping ways it was essential to first provide an especially rigid mattress head. This was formed of a double line of heavy hardwood poles laid across the inclined ways parallel to and just above the up-stream edge of the barge for the full width of the mattress, the individual poles lapping 10 to 15 ft. where they were spiked together; the two lines of poles were also wired strongly to each other. Upon these poles and at right angles to them, and therefore lying between the frames of the sloping ways, were fastened the butt ends of the weaving poles of the mattress. Upon the latter and directly above the first set of hardwood poles was placed another of similar arrangement, and the whole was thoroughly bound with wire. Several head lines were led from shore anchorages under the mooring barge to this mattress head to which they were strongly attached, and thence they extended a considerable distance into the body of the mattress in order to secure a more effective fastening. The weaving poles were fairly straight young trees, preferably willow, which were in the vicinity of 30 ft. long and 5 in. in diameter at the larger end; they were well trimmed of branches and irregularities which would retard the weaving, and were supported in their position close beside the way timbers. The brush which was woven about the poles consisted of willows at least 25 ft. long and from 2 to 4 in. in diameter at the butt. As in the case of other materials used, the brush was passed to the work from supply barges lying at the down-stream side of the building barge.

The weaving poles were spaced from 6 to 8 ft. apart, depending upon the size of the mattress to be built. The men receiving the brush wove it alternately above and below the successive poles in a way to bring a pole first above and then below adjacent lines of brush; the small, flexible ends were always left projecting above the upper side of the mattress, the thickness, stiffness and strength of the whole structure depending considerably upon the length of the small end not utilized in the weaving and the thoroughness with which the brush was driven and held close to the previous course. The butts were placed in the same direction throughout a strip about 5 ft. in width, and the ends were reversed in each succeeding strip as woven. It was often found advantageous to construct the mattress head and the first 20 or 25 ft. of weaving while the barges were lying parallel to shore, swinging them into their transverse position just before launching this finished portion.

To prevent the completed length of mattress from sliding pre-

maturely and to check it when its down-stream edge had just cleared the building barge, several lines of 1½-in. rope were employed to hold it to the latter. Slip lines of the same size and of ample length for use in the final sinking were looped around the mattress head at a spacing of about 12 ft., and their ends were drawn tight and fastened to timber heads of the down-stream edge of the mooring barge. When the weaving had progressed to cover the available width on the ways the lines holding the two barges together and those holding the mattress in position were both gradually slackened, allowing it to slide into the water as the building barge dropped down-stream, while the slip lines held its head from sinking below the water surface during subsequent construction. The mattress head lines, already mentioned as extending to shore, prevented its moving down-stream with the building barge. The movement of the latter was stopped as the down-stream edge of the mattress just cleared its up-stream edge. Then another set of weaving poles were spliced to those nearly covered, with a lap of 5 ft., by spiking and a double lashing of wire, and the weaving then continued as before.

When three shifts of mattress had been thus launched into the water the top grillage was begun. Its purpose was the further strengthening of the mattress and the formation of compartments to contain the stone ballast. It consisted of a line of poles directly above the weaving poles and wired to them every 4 ft., above which lines of transverse poles were wired at a spacing of 8 ft. for the first dozen courses and double this for the remainder of the mattress. As the work progressed small transverse wire cables were attached every 16 ft. of length, and anchored to the shore; these extended across the entire width of the mattress with frequent fastenings. There were also longitudinal wire cables, likewise employed to reinforce the mattress, which extended throughout its length and passed around its head and foot. They varied in size from three-eighths to five-eighths of an inch in diameter, and were spaced at distances varying from 30 to 45 ft., the larger spacing and sizes being the farther from the shore; they were wired and clamped to the body of the mattress at intervals of 16 ft. while under a longitudinal stress to hold them perfectly straight.

The ballasting of the mattress was begun when several hundred feet in length had been woven. This was accomplished by bringing barges of stone alongside from which runways of heavy plank sloped down to the mattress and thence across it. Men wheeled the stone over the runways, distributing it evenly until all but the grillage was

under water. When the desired length of mattress had been completed the sinking of it was begun at the upper end, as illustrated in Fig. 65. A barge of stone was placed at the outer corner, close to the mooring barge, and from it the rock was thrown onto the adjacent portion of the mattress until it was only prevented from sinking by the support of the slip lines. The latter must be slackened somewhat to allow the stone barge to advance across the head of the mattress, and when the whole width had been properly weighted all the slip lines simultaneously dropped the mattress head and it was sunk to

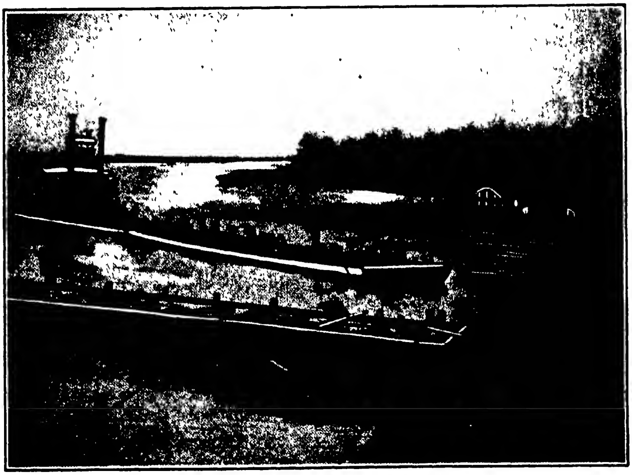


Fig. 65.—Sinking a completed mattress.

position upon the sloping bank. The remainder of the mattress was sunk progressively down-stream, as indicated in the last figure.

Although woven mattresses had been successfully developed from those of small size of the earlier experiences to those of adequate extent and strength in the later years of their construction, experience developed several objectionable features inherent in them the most serious of which was the fact that this type was not compact enough to prevent a considerable scour, through the openings existing in it, in many places where used. Attempts to correct this difficulty by weaving a less length of willow and so leaving a greater length of the flexible branching end to spread over the top, or by adding an extra layer of brush, were found to produce a mattress lacking in the flexi-

bility necessary to allow it to fit the irregularities of the bottom as it should, especially at its outer edge where some undermining is unavoidable, but soon ceases if the mattress is not too stiff to bend and continue to cover the bottom. Consequently their construction was abandoned on the lower Mississippi about 1893, when the stronger and more solid but considerably less rigid fascine type had been found to be much more effective for general purposes, and but little greater in cost.

Omitting the many variations of method of construction tried at different times, especially in the early years of experiment, it may be

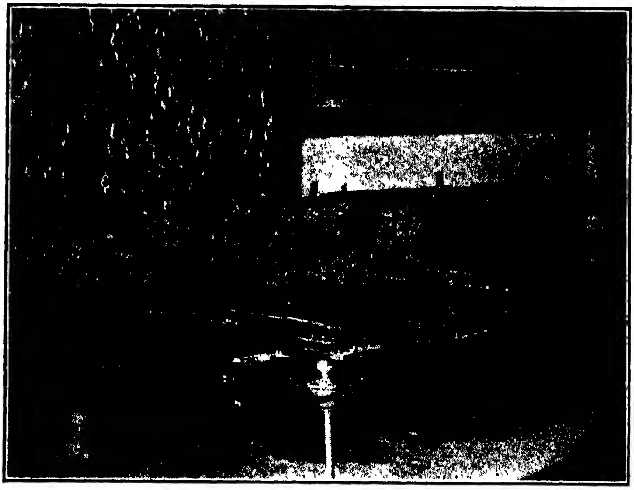


FIG. 66.—Constructing a fascine mattress.

stated that the standard form as developed is indicated in Fig. 66[1] which represents the building of a mattress 300 ft. wide. The general arrangement of barges during construction is similar to that already described as employed for the woven mattresses. The two mooring barges are seen at the right, just below the quarter boat at the extreme edge of the view, lashed end to end and held normal to the bank by cables. Extending down-stream from them is a completed portion of the fascine mattress floating on the water with its lower edge resting upon the inclined ways of the building barges. Immediately below

[1] Photograph of a model in the Civil Engineering Museum of Washington University.

the latter are the supply boats loaded with brush to be used in extending the construction.

The mattress head for the fascine type consists of a bundle of hardwood poles, which individually are from 5 to 8 in. in diameter at the butt and are laid to break joints, all thoroughly bound together by wire strands and thus forming a continuous cylindrical beam having much strength and some flexibility. From the places at which the head lines were attached there extended cables diagonally onward into the mattress to a firm attachment to another similar, but smaller,

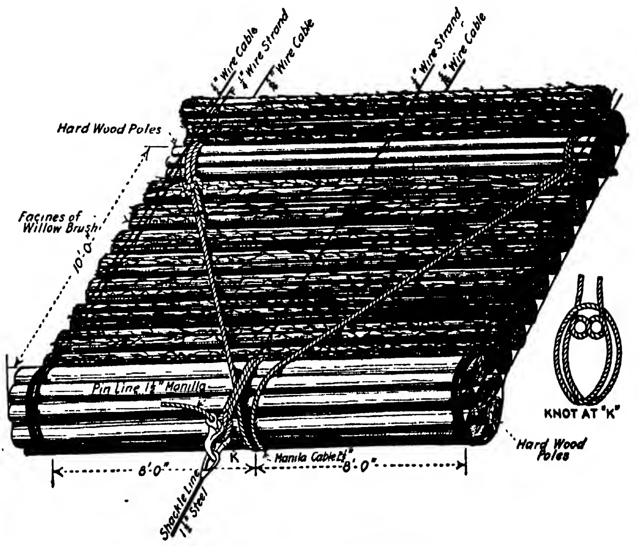


FIG. 67.—The mattress head.

bundle of hardwood poles about 10 ft. from the first, the intervening space being occupied by fascines such as were used in the general construction of the mattress, as shown in Fig. 67.[1]

The fascines are made of brush preferably not larger than about 3 in. in diameter at the butt, each forming a continuous element whose length is equal to the width of the mattress and whose direction is perpendicular to the margin of the river. They are formed at the building barge by placing the brush in such a way that the willows everywhere overlap and have their tops lying a part in one direction and a part in the opposite direction so as to form a fascine of equal

[1] Report, Chief of Engineers, U. S. A., 1894, p. 2918.

strength and size throughout. The brush thus loosely placed is choked, by a chain and lever device, into a fascine about a foot in diameter, and bound by No. 12 steel wire at intervals of about 8 ft. It is then slid to its position on the ways alongside the preceding one which has already been attached to the work.

Extending throughout the length of the mattress beneath the fascines, at a distance of about 8 ft. apart, are wire ropes of a half-inch diameter in the part farthest from the bank and somewhat smaller elsewhere, where the stresses are less. A new fascine is bound to that part of the mattress already formed by means of quarter-inch weaving strands made of steel wires spaced 8 ft. apart also, each of which is passed forward over and around the new fascine, crossing the wire ropes just referred to as it passes underneath, and being brought to the top again between the new and the preceding fascine. It is then engaged by a special device which draws the weaving strand very tight and which compacts the fascine still more, and is then temporarily fastened by a staple to hold the stress thus secured. The next fascine received is similarly bound alongside the preceding one, and this procedure is continued throughout the length of the mattress, a launching being effected whenever the ways are full by dropping the building barge down-stream until the working area is again clear. The weaving strands and the wire rope underneath are clamped together vertically at distances of 10 ft. apart, and both are fastened by staples to the larger brush of the fascines at frequent intervals to increase the thoroughness with which the parts are bound together. When especial strength is necessary it is secured by the use of additional wire ropes, similar to those underneath, placed longitudinally on top of the fascines at a spacing of 8 or 16 ft., and thoroughly fastened to the body of the mattress.

Halfway between the lines of weaving wire, and therefore 8 or 16 ft. apart and extending parallel to the margin of the river, there are bound to the fascines lines of poles which serve the principal purpose of holding the ballast from sliding off the mattress. At the lower edge cross poles are placed at the same intervals and on top of these are usually three lines of longitudinal poles directly above the three outermost lines of the lower layer, all lashed together and thus forming a cribwork to more effectively accomplish the same purpose. In concave banks where there may exist currents of high velocity it sometimes becomes necessary to place a similar construction transversely to guard against the stone being washed down-stream off the mattress. It has been customary to use silicon bronze wire for the

lashing of the top poles so that they will bind the parts of the mattress together after the steel wires have been destroyed by corrosion. But this would not prevent another difficulty, that of the cutting of wire by the sand in suspension, an action which is occasionally rather rapid and severe. However, the general silting of the mattress and its weight of stone are the principal factors in securing its permanence in position. The procedure in ballasting and sinking fascine mattresses is generally similar in its principal details to that already described in the case of the woven kind. The rate of construction of such a mattress is in the vicinity of 100 ft. a day.

The third type, known as framed mattresses, avoids the objections to metal fastenings just mentioned by giving opportunity for a large use of wooden pins in the connections of its essential parts. It also has the advantage, sometimes great, of being capable of being towed considerable distances from a convenient locality for construction to the place where it is to be used. When this is done the mattress is usually framed on ways built on shore, from which it is launched when completed, and a usual size is a length somewhat more than 100 ft. and a width corresponding to that desired and so varying from 100 to 300 ft. or more. On reaching their destined location they are ballasted and sunk so as to overlap 10 ft. or more in order to secure undoubted continuity, as is customary with the other types also; or else the sections are so made that they may be fastened together by wire lashings, and then are sunk.

In their construction framed mattresses typically consist of a bottom grillage composed of poles or sawn timber, each piece having a sectional area of 16 to 24 sq. in., their spacing in each direction being usually about 8 ft. and the points of intersection of the two layers being fastened together by spikes, wires and wooden pins. Vertical posts of a length equal to the thickness of the mattress are placed at each point where the timbers cross each other, and have their lower ends securely fastened to this grillage; this is often done by using timbers of half size in pairs, placed alongside and only far enough apart to allow the stanchions to extend between them in order to make the connection more effective. Upon this bottom frame is spiked a continuous layer of brush, at right angles to this and just above is another course, and a third layer rests upon the second with its brush placed parallel to that of the first. The thickness of each course is ordinarily made enough so that, when compacted, it will be 3 or 4 in. thick. Upon the upper layer of brush there is constructed a timber grillage similar to that

at the bottom; the brush packing is strongly compressed by bringing the upper and lower frames as close together as is practicable by using a special contrivance devised for this purpose; the vertical posts engage the upper frame in a way similar to that provided for their lower ends and are then made secure by spikes, wooden pins and wires. A framed mattress made in this way is also about a foot thick. Sometimes rods and cables are added to strengthen the

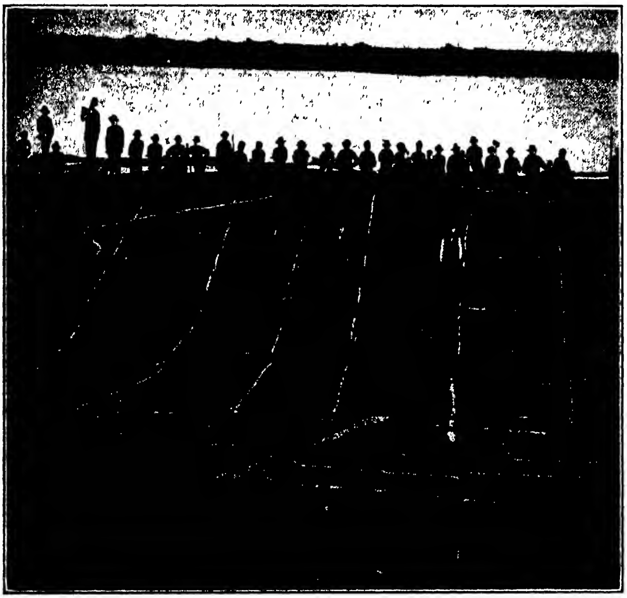


FIG. 68.—Building a connecting mattress; front view.

structure. Poles are then lashed to the upper frame to hold the rock in position during and after sinking, as in the case of woven and fascine mattresses, as well as to give it greater strength.

In many cases mattresses have extended up the bank to the edge of the water at the stage existing when they were placed, and sometimes even higher. This practice exposed the structure to disintegration caused by the decay of the wood when it was uncovered at lower stages of the river. It is customary to fill this intervening space between the channel mattresses and the upper bank paving by relatively small connecting mattresses of similar construction whose

disintegration does not endanger the integrity of the principal parts of the revetment, and which can be replaced at comparatively slight expense. A front view of a woven mattress of this kind is shown in Fig. 68.

The weight of stone required to sink and hold a mattress depends somewhat upon the type used, but more upon the condition of the material composing it with regard to its greenness or dryness, and upon the current conditions existing at the locality at times of greatest duty. It seems to range between 10 and 30 or more

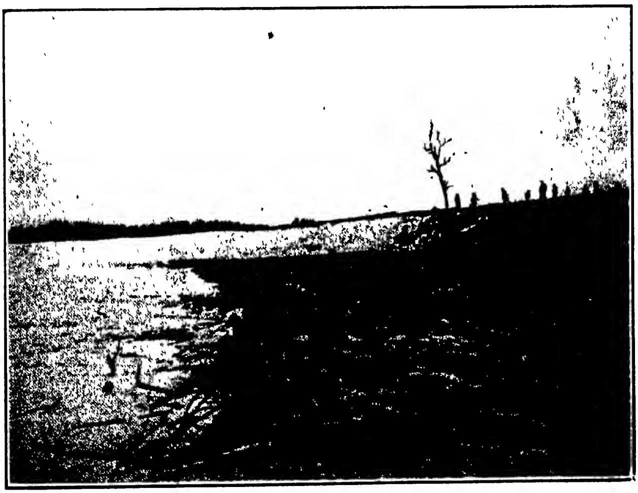


FIG. 69.—Constructing a mattress on a frozen river.

pounds per square foot of mattress surface, and appears to ordinarily average 15 or 18 lb., but is often made twice as much or more at the upper edge for protection against ice or drift. On similar work in Russia, where the mattresses have been made about 20 in. thick, the layer of stone is about 8 in. deep.

Of course many variations from the typical methods described have occurred in mattress construction due to differences of conditions under which the work has been done and of the duty required of them. Not infrequently they have been constructed so that the direction of the fascines has been parallel to the bank instead of normal to it, because in this position they offer a greater resistance to the sliding of the rock ballast if the slope is steep, and they seem

to fit the irregularities of the slope somewhat better. Piles are often used to a greater or less extent to aid in the control of their correct placing as they sink, as well as to assist in holding them in their final position. On the upper part of the Mississippi River, where the mattresses need to be only 20 to 60 ft. in width, not only does the work allow a relatively simpler plan and equipment, but some of the details are altered; such as the making of the fascines only about 20 ft. in length and lapping them so as to break joints in successive courses, the inclining of the ways to a slope of 25 or 30 degrees for the fabrication of the narrower mattresses or designing them to tilt when ready for a launching, and the considerable use of lath yarn for binding material. On this river, as well as on the Missouri and others where the ice formation is heavy and reliable, it has sometimes been found an economical proposition to build the mattresses on the surface of the ice which is then cut to allow their sinking into place. This method of construction is shown in Fig. 69 (p. 255),[1] which also illustrates a diagonal arrangement of the weaving of the brush. Framed mattresses, in places of severe duty, have been sometimes made with four or five layers; and occasionally with less than three, where their exposure was not great. Cottonwood and other pliable brush has been used when live willow has been difficult to secure. Solid concrete blocks, brick, etc., have been used for ballast in places where stone is quite expensive. For this purpose the concrete is cast in molds of advantageous dimensions and afterward is broken to convenient sizes. As a satisfactory quality can be made in which the weight of cement is only 7 or 8 percent of that of the aggregate, the cost need not exceed $2 per ton.

The progressive development of mattress construction has had constantly in view their increase in durability and strength to meet the conditions of service which experience has shown to be necessary, and at the same time the consideration of economy in expenditure has been an essential feature. The shore connections have been improved to more adequately meet the severe duty required of them, the binding together of the different parts has been made more effective, and the details of construction have been so modified from time to time that the danger of a local injury extending so as to cause the destruction of an entire mattress has been minimized. That the possibility of such injury is not remote may be readily realized when one recalls not only the persistently severe activities of swirling currents of high velocities in concave banks, but also such

[1] Report, Chief of Engineers, U. S. A., 1901, p. 398 of Supplement.

dangers as those of floating trees and drift and of ice to those parts of revetment near the water surface. An illustration of the latter is shown in Fig. 70.[1] Next to the upper portion of a mattress, it is generally the case that the lower edge is subjected to the most severe attack. This results from the scour of the unprotected earth adjacent, which is liable to undermine the outer edge. Sometimes a heavy stone paving has been laid from the edge of the mattress outward to prevent this action, as on the Weser River. In this country it is

FIG. 70.—Ice gorge at a revetted bank.

customary to deal with the situation in a different manner; that is, by extending the structure to the bottom of the under-water slope so as to minimize the opportunity for erosive action, and at the same time to make the mattress flexible enough to settle as fast as undermining occurs, and thus to soon terminate its progress.

Fascine mattresses are the kind generally used on the lower Mississippi River where operations are of unparalleled magnitude. They have been experimentally developed in a most creditable way until they may be characterized as having proved successful under conditions of construction and service of especial difficulty. They are the greatest in size that have been built, often being 350 ft. in

[1] Report, Chief of Engineers, U. S. A., 1910, p. 1826.

width and 1000 or 1500 ft. long, and placed in water from 50 to 100 ft. in depth, which often has a velocity as great as 6 or 8 ft. per second. Yet the magnitude of this work is so considerable that as yet only the more important points have been protected; hardly 70 miles of bank in the whole length of the lower river have so far been revetted. On the contrary, the fascine mattress work of the upper river is so comparatively simple and inexpensive because of the relatively more stable soil, moderate velocities of current, small depths and consequent reasonable widths of mattresses that the length of its revetted banks is more than three times as great.

Framed mattresses are still used to a considerable extent on the lower Mississippi River; of the Albemarle Bend revetment above Vicksburg, constructed in 1912, a part was of the framed type and the remainder consisted of fascine mattresses in order to test the comparative advantages and disadvantages of each under similar conditions. While woven mattresses have long been abandoned for the general revetment work of the lower river they are still extensively employed on other streams, such as the middle Mississippi and the Missouri Rivers.

Occasionally logs and other timber have been used for this kind of construction. Many miles of bank on the Missouri and the middle and upper portions of the Mississippi River have been protected under water by lumber mattresses where brush is limited in quantity and therefore expensive. A standard design is illustrated in Fig. 71.[1] They are made of cheap cull lumber which, however, must have no defects of such size as to seriously weaken it. While some have been made by placing the boards in two distinct layers and nailing them where the pieces of one course cross those of the other, it is preferable to adopt the method of weaving the elements in much the same way as has been described for the woven brush mattress, as indicated in the illustration. The size of lumber used throughout is 1 in. thick, 4 to 6 in. wide and not less than 12 ft. long. From firm fastenings in the head-block, made especially strong as shown, extend the weavers whose spacing is close enough to provide that the shortest board will engage at least four of them. Upon these, and therefore parallel to the head-block, are woven the strips of board with open spaces between them somewhat narrower than the minimum diameter of ballast that will be used to sink the mattress. Stringers and cross binders are added on top to strengthen the structure and to keep the rock from sliding off; they are formed by

[1] Report, Chief of Engineers, U. S. A., 1901, p. 2220.

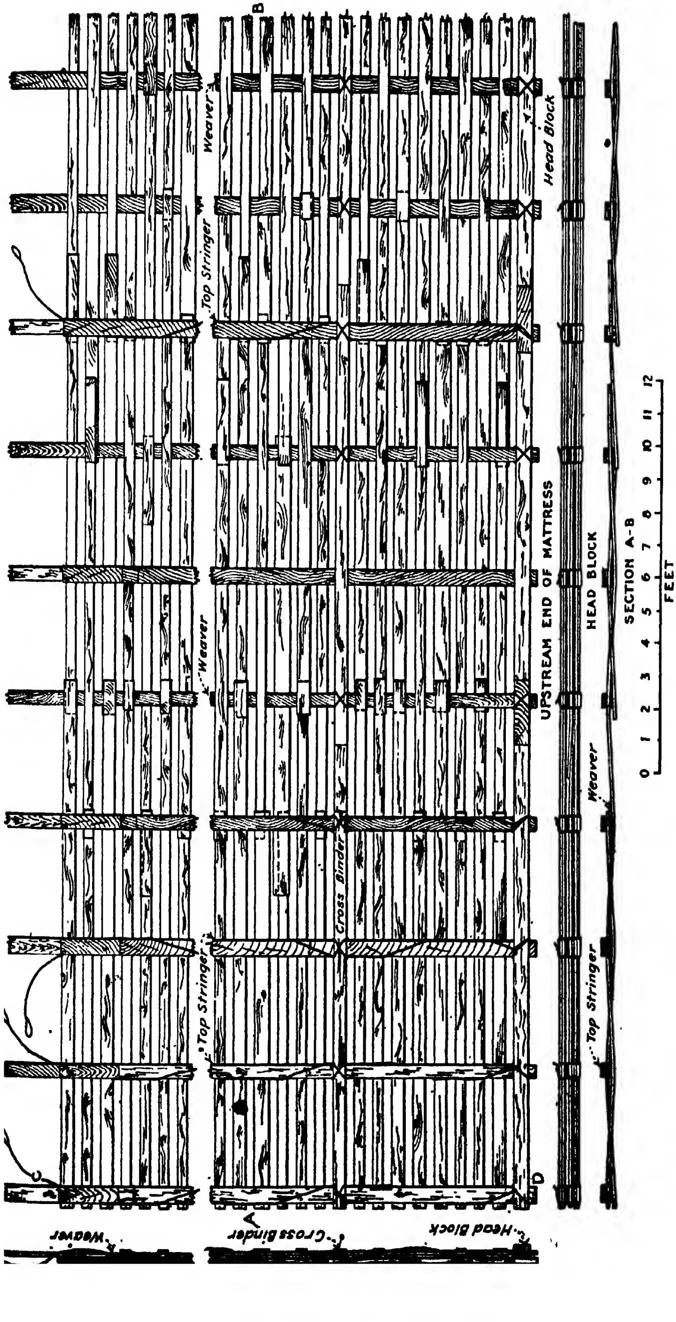


Fig. 71.—A lumber mattress.

nailing two or more boards together to form the necessary thickness. All joints are also spiked and wired in order to make the mattress as strong and rigid as possible, but it is so thin that it is very flexible and in some danger of breaking in launching and sinking; therefore the cables are bound about the framework at frequent intervals, as shown, if the mattress is large.

The general procedure of building, launching, ballasting and sinking lumber mattresses is similar to that used for those made of brush. The rate of construction is much more rapid, the necessary plant is less extensive than that needed in the construction of brush mattresses, and a smaller quantity of ballast is required to sink them. The experience with lumber mattresses has in general been favorable, especially where live brush is expensive to obtain in the desired quantities and when placed in rivers where the velocities are not excessive or the soil too easily eroded.

66. Grading and Paving the Bank above Water.—The treatment of that portion of concave banks which is above the surface of the river necessitates its initial grading to a regular slope which will be permanent, because the action of the eroding currents usually keeps it so steep that it is in an unstable condition. The required slope varies with the character of the earth composing the banks, and should not be steeper than the slope of repose of the soil to secure its permanence, nor much greater than this in order to avoid the unnecessary expense of an excessive amount of grading. The latter consideration has sometimes led to the placing of revetment on banks so steep that their subsequent sliding has destroyed the protective work placed upon them. It is often economical to plan the adequate drainage of such banks as may become seriously saturated with water at any time, in order to secure for them a steeper slope of repose than they would otherwise have and so diminish the amount of necessary work. The required slope is usually between 1 on 2 and 1 on 4 and is very frequently 1 on 3; although earth is sometimes found of such stable character that it will stand as steep as 2 on 3, as in many places on the upper Mississippi, Volga and Dnieper Rivers.

The grading may be done either before or after the placing of the mattresses opposite, and often the decision of this question rests upon the time at which the equipment is available for either part of the work and the ability to organize the mattress construction and the bank work in a way to avoid interference and to expedite the progress of both. If the sequence of operations is not controlled

by such considerations, and when the spoil is wasted into the river, it is believed that the advantages of uniformity of under-water slope of the bank and the usefulness of the earth for filling and loading the newly completed mattresses will place the grading subsequent to the finishing of the adjacent sub-aqueous part of the revetment.

Grading with spades and shovels has been occasionally the method used, in cases where its extent is limited; but ordinarily such hand tools are utilized only for the small amount of the final shaping of the slope. Drag and wheel scrapers and other kinds of machinery used in earth-

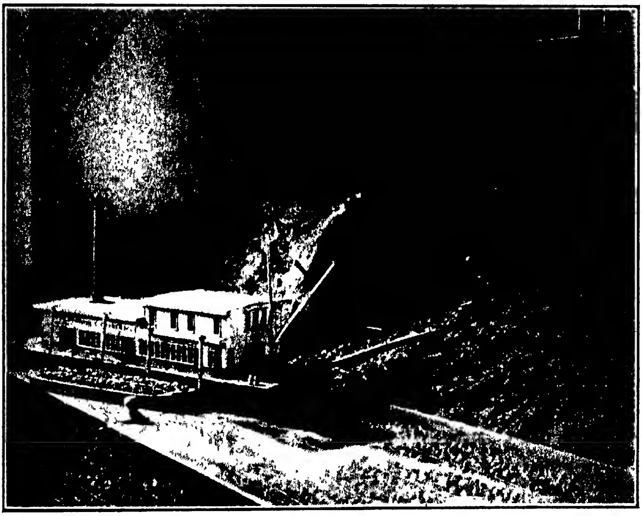


FIG. 72.—Hydraulic grading of a river bank.

work are quite generally employed for the purpose when the equipment for sloping the bank by a powerful water jet is not available. Hydraulic grading has been used increasingly for more than thirty years and, being by far the most effective and economical method known where large quantities are involved, it now constitutes the standard for extensive operations. The machinery is mounted on a special boat and consists of the pump and accessory mechanical equipment necessary to supply the water in a quantity of perhaps 2000 gal. per minute and under a pump pressure usually in the vicinity of 150 lb. per square inch. The water is drawn from the river through suction pipes about a foot in diameter, and large discharge

pipes lead from the pump to the forward part of the boat where several lines of large hose are attached for use in the hydraulic grading. The slope is carried forward as uniformly as is practicable, as shown in the right-hand portion of Fig. 72 (p. 261),[1] and the steep bank, rising 40 ft. above the water surface, is reduced by directing the jets against the bottom at the plane of the desired slope and so under-

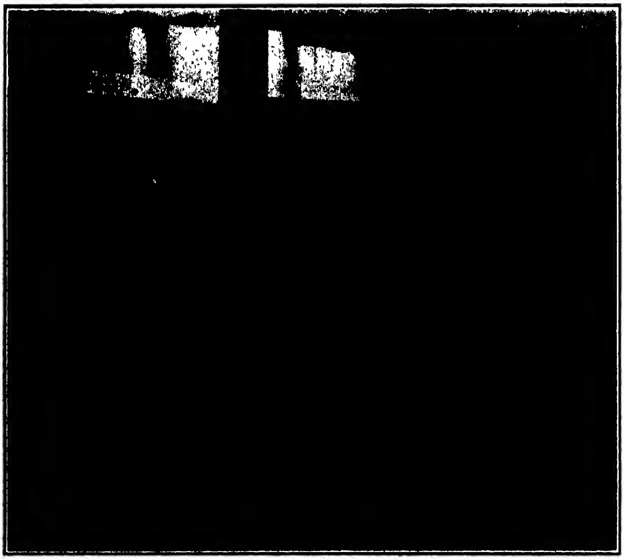


FIG. 73.—A completed revetment.

cutting it. It is usually advantageous to keep the work at the top of the slope somewhat in advance of that where the cut is deeper so that the volume of water flowing to the river shall assist in softening and undermining the whole width of the excavation and help to wash the caving mass of earth into the stream. Several thousand cubic yards of earth may ordinarily be removed in a day by the hydraulic method and the field cost of grading in this way is reduced to an average of from 3 to 7 cents per cubic yard.

The grading is followed at once by the revetting of the upper bank. Experiments in the earlier years with brush or fascines, wire netting,

[1] This and the following illustration are photographs of models in the Civil Engineering Museum of Washington University.

etc., proved unsatisfactory because of the rapid decay of wood and the corrosion of metal. Consequently the standard material used for this work has been stone for many years. It is now a quite general practice to cover the sloped bank with a foundation course of a fine material, such as quarry spalls or gravel, to a depth of 3 or 4 in., which secures a greater compactness than is possible with larger stone and so reduces the danger of local erosion by the searching swirls of high water currents. Upon this is laid by hand a stone riprap usually 6, 8, or 10 in. thick, the depth depending upon the severity of the exposure. The thickness of stone revetment on the banks of the Volga is as much as 20 in.

A typical illustration of a completely revetted bank of the lower Mississippi River is shown in Fig. 73, in which the low water surface is represented by the horizontal strip of glass across the middle of the slope. Below this is seen the fascine mattress which slopes about 300 ft. downward to the foot of the under-water bank; and above is represented the stone revetment reaching from the water's edge to the top of the immediate bank, an inclined distance of about 100 ft. At the extreme back of the model is seen another horizontal glass strip which represents the high water plane at the levee on which it is placed. The depression between the levee and the top of the immediate river bank is due to the removal of earth for the construction of the levee.

67. **The Use of Concrete for Revetment.**—The use of concrete for the protection of the upper bank has been tried rather extensively. For this purpose it is usually cast in place and divided into rectangular blocks. Their usual form is that of transverse strips, 6 or 8 ft. wide, with their length equaling the width of the sloped bank. The slabs are usually made 4 in. thick. Apparently the principal cause of any deterioration or lack of permanence of this kind of revetment is not due to surface attack so much as to the loss of the earth support underneath, which sometimes occurs. Perhaps this situation is largely due to the restraint which the practically impervious slabs oppose to the percolation of the ground waters to the river, so encouraging their ultimate concentration into streamlets under the concrete, which produces a slow undermining. There seems to be at present, in this country, no great preference for the use of concrete in this way except in special cases or when stone of the required quality and size costs more than about $2.50 per cubic yard. In recent years some banks have been protected above water by concrete of the same general thickness, but reinforced

with steel rods. This construction seems to promise a greater effectiveness and it may have a considerable use in the future in places of particular exposure or importance, or where good stone is quite expensive.

Concrete has also been used for the protection of the under-water slope. Considerable experimental work of this type has been prosecuted in Japan, especially on the Yubari and Ishikari Rivers, with evident success. For this work the concrete blocks were made 6 in. square and 2 ft. long of 1:3:6 Portland cement, sand and gravel, and reinforced near the long edges by four pieces of No. 12 galvanized wire bent inward at their ends. To build them into a mattress upon scaffolding above their destined position metal bars, 6 to 10 ft. in length, $1\frac{1}{8}$ in. wide and $\frac{3}{8}$ in. thick, were placed overlapping each other in a continuous line to form its lower edge. Through holes spaced 1 ft. apart in these bars, No. 4 galvanized steel wires were threaded and extended at right angles to the bars, and upon those wires the concrete blocks were strung by means of two holes formed in them when the blocks were cast. As these holes were 6 in. from the ends of the blocks and the blocks were laid to break joints, each pair of wires passed successively through the two holes of a single block and then through the adjacent holes in the abutting ends of two different blocks. Thus a continuous and flexible mattress can be woven of the width and length desired, the latter being preferably equal to the length of bank to be protected. It has also been customary to make these mattresses rather narrower than may be necessary in the hope that the resulting scour and settlement at the lower edge will shortly cease in the securing of final stability, thus involving less expense than if they were built of the full width necessary to prevent any scour; and if stability does not soon occur, the width is then increased. Mattresses have been thus constructed of different widths, from 12 to 93 ft., and in depths of water varying from 4 to 40 ft. They have also been subjected to floods, freezing weather and the severe action of ice and floating drift without material injury.

The conclusions state that "the writer believes his reinforced concrete mattress to be the most economical, durable, flexible, and altogether effective bank protection and that it is particularly adapted for the protection of those river banks where caving is most severe. The setting of the mattress may be better made at first with a slight width, as the best economy may be accomplished in this way when only so much of the mattress width is added bit by bit as is really necessary for the site in

question, thus avoiding the costly setting at first of the full width of mattress down to the bottom of the thalweg."[1]

Concrete revetment below water has been but little used in this country. So far it appears to have been employed only to protect the most vulnerable part of the bank defense, being that zone extending from the foot of the upper bank protection to a level below the lowest stage to which the river falls. This strip is especially difficult to control because the paving of stone or concrete can be carried downward only to the surface of the water existing at the time of the

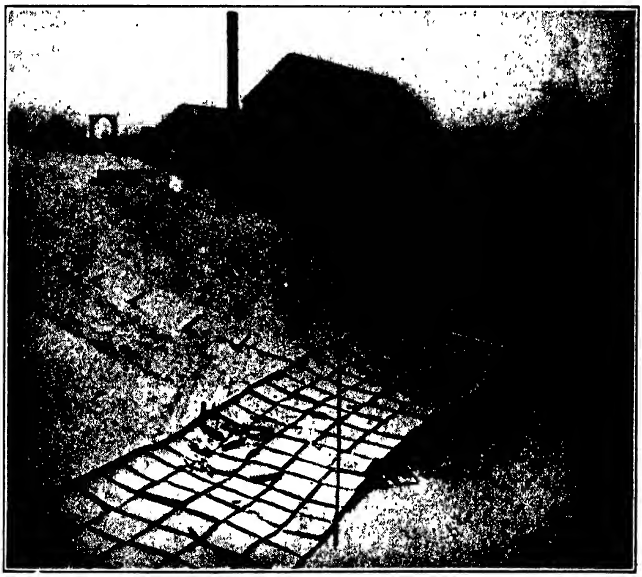


FIG. 74.—Reinforced concrete revetment.

work; and when the connecting (or the river) mattresses are placed to join this edge, their upper margin is not only exposed to the danger of floating drift and ice thrust but is also subjected to periods of alternating submergence and exposure to air because of the varying stages of the stream. Consequently it has been proposed to fortify this zone of especial weakness by a system of reinforced concrete blocks. In the last few years this has been tried at several places on the Mis-

[1] Engineering News, Vol. 67, p. 922, and Vol. 69, p. 513.

souri River and its tributaries and gives promise of excellent results. In 1912–13 a thousand feet in length of bank was thus additionally protected by a flexible covering of reinforced concrete blocks forming an apron 10 ft. in width. The blocks were about 2 ft. square and 4 or 6 in. thick, reinforced and tied together by wire and small bars which were galvanized in the case of those not entirely embedded in the concrete. The connecting bars also engaged the lower edge of the reinforced concrete revetment of the upper bank, thus securing a mattress both strong and apparently very durable and resistant. The general appearance of this protection is shown in Fig. 74[1] (p. 265).

68. The Cost and Maintenance of Revetment.—The cost of revetment on the Volga River is given as $15 per foot of bank, but this seems to include very little protection of the upper bank. For the lower Mississippi, including the mattresses varying from 200 to 350 ft. wide, and the sloping and paving of the upper bank which ordinarily involves an additional 30 percent of width, it is usually stated as averaging about $30 per foot of length of river so protected. Upon the lower Missouri and the middle portion of the Mississippi, where the width of revetment is only about one-third as great, the cost is correspondingly reduced. For the still less extensive protection of the upper Mississippi River the first cost for the whole period covered by such construction has similarly averaged hardly $3.50 per foot of bank.

A much more definite method of considering the cost is that in which the superficial area is made the basis. While the expense of fascine mattresses seems to average somewhat more than those of the framed or the woven types, yet local conditions such as the cost of materials and the efficiency of the labor much more than overcome those differences. The material and work of building the mattresses involves an expenditure perhaps one-fifth greater than that of the ballast and the operation of sinking them into place. The general field cost of the standard brush mattresses has usually been between 6 and 9 cents per square foot in place. The expense of those constructed of lumber has often been from 20 to 40 percent less than if made of brush in the same locality. The first cost of the reinforced concrete mattresses of Japan is reported as about 11 cents per square foot, and for those special ones experimentally constructed so far in this country it has been considerably greater. The standard stone paving of the upper bank varies more in its initial unit cost than do the brush mattresses, but the average is about the same. The

[1] Report, Chief of Engineers, U. S. A., 1912, p. 2196.

relative expenditure for a concrete covering above the low water surface of course depends upon the local conditions. In some localities it is both cheaper and more durable than stone riprap. There are instances where a 4-in. concrete protection has been constructed at a cost of about 5 cents per square foot; but as a rule the concrete is the more expensive, especially if reinforced.

It is practically impossible to state the charge which should be made, in addition to the initial field costs summarized above, in order to represent the outlay for deterioration, repairs and care of plant when not in use, and for the prorated share of office and other administrative expenses, capital charges and other similar general expenditures indirectly involved in the construction of revetment. Various scattering estimates range from about 20 percent additional to 60 percent or more. Possibly a general average would show a total expense in the vicinity of 40 percent in excess of the field costs, which are ordinarily reported.

The question of the average charge for the maintenance of revetment is also a very difficult one, both because it is often omitted or is obscurely referred to and for the reason that different classes of construction exhibit so great differences, especially in localities of varying exposure and in years of so great diversity in the severity of the service of the protecting works. It is generally noticeable that the more adequate materials and the more thorough workmanship effect a reduced expenditure for maintenance and repairs; but the result is a greater first cost. The comparatively favorable regimen and the character of the revetment constructed on the upper Mississippi have combined to exhibit a quite low outlay for maintenance, which amounted to about 6 percent of the first cost during the first 35 years of construction there. On the contrary the magnitude of the structures and the severity of their service on the lower Mississippi have resulted in an average expenditure for maintenance variously given as from 2 to 5 percent per annum. The experience of forty years on the middle Mississippi River indicates that the cost of maintenance of mattresses has been only about two-fifths as much as that of the upper bank protection, in proportion to the original outlay upon each; the expenditure during that period upon repairs and maintenance of both kinds of revetment has been 13½ percent of the first cost, and the proportion destroyed in that time has been 29 percent of the total amount constructed;[1] and therefore, if it be considered that both items should be combined in arriving at a true

[1] Report, Chief of Engineers, U. S. A., 1912, p. 2137.

charge for maintenance, the result is an average of a trifle more than 1 percent per year, apparently exclusive cf capital charges. It is stated that the outlay for maintenance of bank protection in Russia has been small.

There is an auxiliary defense of the upper bank that grows more effective with its age if it is properly trimmed and cared for, which has been employed in continental Europe particularly in connection with the work of maintenance. It is the planting of willows, or other similar growth, thickly among the bank-protecting materials laid above water. These gradually form a network of interlacing roots which firm the soil, especially when it is saturated and in need of aid in resisting the settling of individual stones of the paving, and develop a pliant mass of tenacious branches against which the disrupting effects of the high water currents are largely cushioned and harmlessly fended off. Willow planting has thus been used on such rivers as the Neisse, Bober and Queis; and also in Japan, where the thickness of the riprap was reduced to 5 in. when laid in panels 3 ft. square, defined by horizontal and transverse rows of planted willows whose vigorous growth completed the effective protection. Because of its tenacious and rapid growth, Bermuda grass has been successfully used abroad for the same purpose in places of moderate exposure.

CHAPTER VIII

DREDGING

69. The Frequent Serviceability of Channel Excavation.—It is often advisable to directly remove by mechanical means the obstructing silt, sand, clay, or rock which form the usual impediment to the navigation of rivers and so limit their navigable depth. This procedure may be either a temporary expedient or a permanent method of improvement. When a reef or any other rock formation is encountered in the bed of a river, its resistance to the erosive action of the current is so pronounced that it very frequently constitutes a prominent obstruction in the boating channel; but, when once removed, it will not again hinder navigation. The same is often true in the case of tenacious clay deposits. Less cohesive earths, such as silt or sand, are the material characteristically forming the objectionable bars in waterways, and their elimination is the ultimate object of river improvement. If this is accomplished by regulation it is frequently true that the original removal of the bars must be effected by dredging because the current is too feeble to erode them, even when sufficient in strength to prevent their subsequent recurrence. The alternative procedure is to omit the works of contraction and thus adopt a method of improvement which requires not only the original excavation through the bars but also the periodic continuation of this process as often as they may form again. In most rivers the latter method would result in economic waste; but in alluvial rivers whose commercial service is entirely problematical or in those of notably unstable regimen whose size is so great as to make the cost of contraction comparatively excessive, it is best to rely upon the excavation of shoal places, repeated as frequently as they recur, for the maintenance of the navigable channel.

Sometimes a designed combination of the two systems secures the desired result at a minimum cost, utilizing works of regulation to only that partial extent for which the resulting deepening of the bars is considerable, and dredging the remainder as found necessary. In fact, experience and consultation are gradually leading engineers to more frequently resort to both regulation and dredging as offering

the most reliable and economical way to secure the desired results; advocates of regulation realizing with increasing definiteness the auxiliary aid and advantages which are available in the efficient types of dredges of the present day for securing the ultimate part of the deepening which is often so difficult and expensive to secure by regulation alone, and partisans of dredging inclining more and more to appreciate the economy and desirability of securing the assistance of regulating works in greatly reducing the volume of material to be removed by dredging when such works may be advantageously constructed to prevent the recurrence of the greater part of the shoaling deposits, which would have to again be excavated. It is the principles and methods of operation involved in the successful periodic removal of obstructing bars of sand and silt which form the subject of consideration in this chapter.

70. Various Devices of Occasional Utility.—To accomplish the removal of the crests of bars, various devices have been employed, and numerous others have been suggested.[1] The first type has for its object the throwing of the sediment of the shoal into temporary suspension by means of drags, scrapers, jets or large, rapidly rotating screws, and depending upon the current of the stream to carry this suspended matter away. This method is hardly applicable to a bar composed of a tenacious material, like clay, nor to a shoal place at which the velocity of current is too slight to transport the material composing the bar. It is also true that only a fraction of the material stirred up by the mechanism is carried down-stream from the shoal, and some of that which is transported by the current is liable to be deposited upon the next bar below, resulting in an increased obstruction there. Yet there have been cases where such devices have given temporary relief; as in the use of cutters fastened to an adjustable framework attached to the bow of a suitable boat, and operated by backing the steamboat down-stream from the upper edge of the bar to the lower, scraping and stirring up the sediment as it proceeded. This method was used on the upper Mississippi River nearly fifty years ago, and deepened the crossings a foot or more. When, later, contraction works were built to permanently secure the increase of depth, the temporary expedient was no longer needed. A comparatively recent employment of the method of raking coarse detritus from shoals into the pools below them has proved advanta-

[1] See paper by J. A. Ockerson, "Dredges and Dredging on the Mississippi River" in Transactions of the American Society of Civil Engineers, Vol. 40, pp. 215–354.

geous under the special conditions encountered on portions of the Columbia and Snake Rivers. It is stated[1] that raking has proved more effective than dredging in deepening many of these heavy gravel shoals. The rake, weighing 5000 lb. and measuring 12 ft. across the points, is shown in Fig. 75, suspended from the dipper boom of the dredge, ready to be lowered into position. It is operated by repeatedly backing the dredge down-stream across the shoal, thus loosening and dragging about 2 cu. yd. of gravel into the pool below at each trip. This procedure is repeated until the desired channel width has been

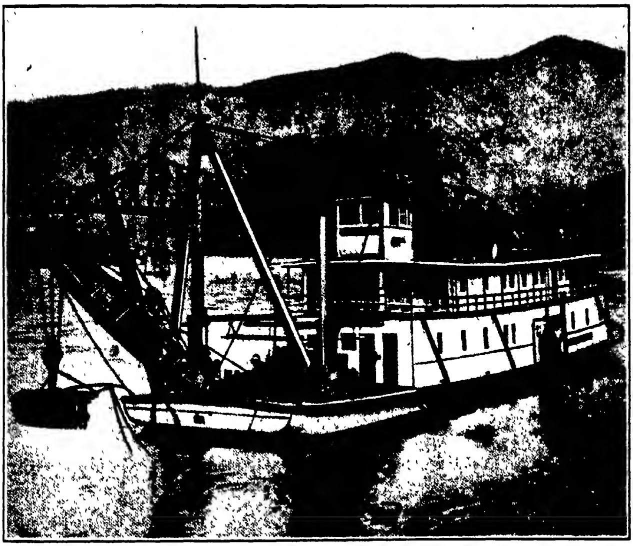


FIG. 75.—Dredge with rake attachment.

deepened a foot or two, which usually requires from fifty to a hundred trips through the shoal. In places where the pool below is not of sufficient depth to receive the waste, it is necessary to substitute the dipper for the rake and to dredge in the usual way. The cost of deepening channels by raking is not given.

Water jets have also been successfully used on various occasions to agitate the sand and so produce its removal from the crest of a bar, both as a temporary expedient and as a permanent outfit for a

[1] Report, Chief of Engineers, U. S. A., 1906, p. 1985.

boat. An example of the former is that of pile drivers, each having a pump of 165 gal. per minute capacity, four of which combined to lower the crest of Horse Tail Bar of the Mississippi River more than 2 ft. in one day, in 1881, their work being mainly accomplished when moving up-stream across the bar. Another instance occurred during the last few years of the nineteenth century when a jet dredge was constructed with a capacity of 10,000 gal. per minute for each of its two 15-in. centrifugal pumps, at a cost of about $18,000; and operated down-stream from edge to edge of the bar, the jets agitating the sediment and carrying it into suspension for transportation down-stream by the current. The cost of its operation was $2650 per month and it succeeded in deepening the channel about 2 ft. in average amount when used on short bars, the variation in the effectiveness of its work being about 50 percent from the figure stated.

Another type of device for securing temporary deepening consists of current deflectors, of which there are two general classes; those planned to deflect the current downward upon the top of the bar, and those designed to produce a lateral deflection and concentration. Of the first sort there have been many inventions differing in detail, but most of them are similar in providing plates, cells, boxes, compartments or other adjustable forms, which are generally attached to the bottom of a boat in a way that is intended to throw the current downward upon the sediment so effectively that an efficient scour will result. While the few actual trials of this principle have produced some deepenings, no case is known where the results were really satisfactory.

To secure a deepening by temporary devices for concentrating the flow of a river at the place where the formation of a channel is desired across a bar, numerous projects have been proposed which vary from a row of trees anchored along the line of the intended deflection to a number of boats which are to be sunk end to end along the line of that barrier, and which are to be pumped out and floated when the expected result has been accomplished. This style of temporary current deflecting and concentrating device bears some resemblance to permanent construction works in regard to the natural forces which are to be utilized for the desired purpose; and to be worth while, the cost of their construction and operation must not be excessive. One of the few devices of this sort which has actually been used was designated a "portable jetty." These were employed for several seasons about fifteen years ago on the Mississippi River between the mouths of the Missouri and Ohio Rivers; but at

the end of that time they were supplanted by the more adaptable and economical service of dredges. As employed, these portable jetties consisted of a row of piles driven 10 to 20 ft. apart to which was horizontally lashed a line of timbers a short distance above the water-surface; in one case small flat boats, which were available, were substituted in place of the longitudinal timbers. These timbers or flats supported the upper edge of a continuous series of corrugated steel plates, each 10 ft. wide (in the direction of the "jetty") and from 10 to 20 ft. long, depending upon the depth of water, placed in a position inclining considerably up-stream, with their lower edge resting upon the bar. It was found necessary to stiffen each of these plates, of No. 14 gauge, with three 5-in. I-beams riveted to them, to attach suitable links to assist in handling them, and to prevent undue scour where they rest upon the bottom by tying to their lower edge a fascine mattress of light construction, about 8 ft. in width, upon which stone was thrown. The line upon which to build this temporary construction at a bar was that which would most advantageously concentrate the current across its crest so as to induce the necessary scouring effect, and everything was removed at the close of the low water season and stored in anticipation of the next season's requirements. The length of such temporary structure at each bar averaged about 1000 ft. and the cost per foot of each season's work ranged from $3.00 to $4.40 per lineal ft. The deepening effected varied from 1 to 3 ft.

Few of the devices, referred to above, have long survived the test of use. In the case of one, certain deficiencies are found; and with another type, different objections predominate; but the difficulties of all are, in general, a greater or less share in a group of adverse characteristics which have been found to accompany projects of the kind, such as lack of sufficient current capacity to remove the sediment; the silt remains in the river, to add to the trouble caused later by the sedimentary activities of the stream; the limitation of additional depth attainable; the expense involved; and the uncertainty of results. Experience has therefore led to the practically exclusive use of dredges for the direct removal of earthy material encroaching upon the required navigable channel.

71. Dredges of the Grapple, Dipper and Elevator Types.—Dredges which have a grapple type of bucket, such as those commonly called the "orange peel" and the "clamshell," have the bucket hung from a swinging boom and operated by two chains or wire ropes, and this sometimes has a capacity as great as 10 cu. yd. The general

appearance of the dredge and its parts, except for the bucket and its mounting, is much like that of the dredge next described. The especial field of usefulness of the grapple type is in soft material at very considerable depths; and for the latter reason particularly this kind is but rarely used for improving the navigability of rivers.

The dipper (sometimes called the shovel, or the bucket) dredge constitutes that type which is the most generally adaptable to all the varied requirements of subaqueous excavation. It is, in effect, practically the ordinary steam shovel mounted on a boat and adapted to conditions imposed by work under water. A typical photograph

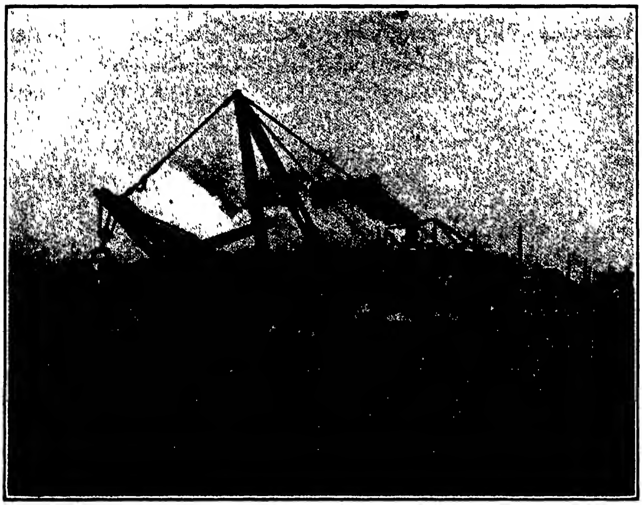


FIG. 76.—View of a dipper dredge.

of a dipper dredge is shown in Fig. 76[1] in which a scow for receiving the excavated material is seen on each side of the dredge, the one deep in the water being already filled and ready to be towed to the dumping place; while a schematic outline of its essential parts is given in Fig. 77.[1] From the latter it appears that a dipper dredge is a machine of real simplicity and directness of action, of positive and definite application of power to the work to be done, and easily

[1] Both of these illustrations, as well as the next two following, are from the paper of A. W. Robinson on "Dredges, Their Construction and Performance," in Trans. Am. Soc. C. E., Vol. 54, Part C, pp. 271–301.

controlled both in changing position and in the performance of its work. The details of its machinery and apparatus have been improved through experience, one of the later advantages being the substitution of wire rope in place of chains and so increasing the speed of operation, reducing friction losses, permitting a more advantageous angle of lead to the dipper, and securing ample warning of the weakening of the cable through wear. The edge of the bucket is left smooth for soft material, but teeth are attached when working in hardpan or loose rock. Its capacity varies from 2 to 15 cu. yds., while the more usual size is now from 4 to 6 cu. yd. Speed of operation averaging less than a minute per dipper load is not rare, and dredges designed for effective work at depths of even 40 ft. are now built. Dipper dredges are especially adapted to excavation at moderate depths and in a material which is particularly hard or tenacious, though they are effective in any except hard rock. Their

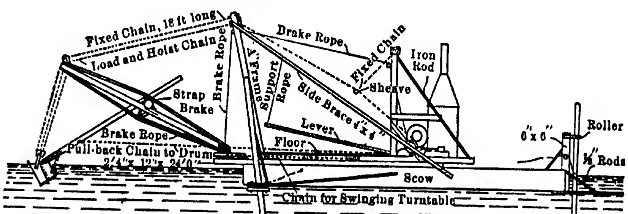


FIG. 77.—Outline of its essential parts.

use in the dredging of rivers is rather limited except near their mouths, but they are sometimes used in non-tidal reaches because of their availability whenever occasional need occurs for their services, or in case of bars to be dredged consisting of gravel, clay or other quite stony or tenacious earth.

A third type, much used in Europe but only occasionally in this country, is the elevator (or ladder) dredge, the distinctive feature of which is illustrated in Fig. 78 (p. 276), in which the buckets are armed with teeth for work in a hard material. The elevator mechanism usually operates through a well opening extending through the hull of the dredge in its middle portion, but is sometimes mounted at the bow and occasionally at the stern to permit the dredging of its own channel; the buckets have a capacity usually between ½ and 1 cu. yd., but they sometimes have twice the latter volume; the ele-

vator frame extends below water to the depth of the required excavation and the dredge is so manipulated that the buckets ascending from the bottom constantly engage the exposed face of the material to be removed and carry it to the top of the frame whence it dumps into an inclined chute through which it slides to the waiting scows in which it is towed to the dumping grounds, or is discharged into the hopper compartments of the dredge itself if the latter is designed to receive the excavated material in its own hull for transportation to

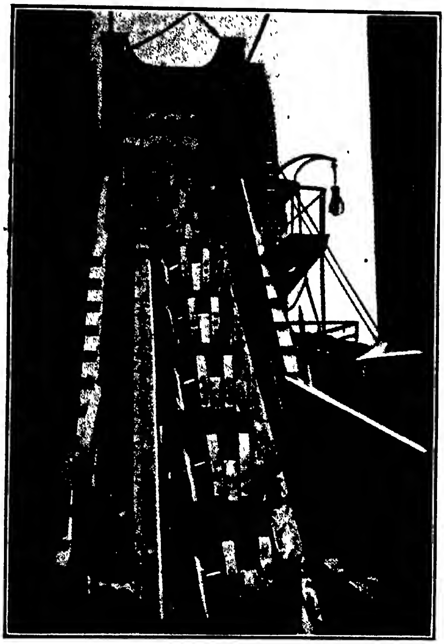


FIG. 78.—Buckets of an elevator dredge.

the place of deposit. Its high cost of construction and of operation require continuous service to effect the dredging with real economy; and with its complexity of mechanism and its need of ample space to operate, it is a machine of rather restricted range of usefulness in comparison with the dipper dredge; and yet this type has a certain field of particular adaptability, marked by circumstances where there are great volumes of material to be excavated, which is of fairly compact quality.

The conditions existing in the St. Lawrence River from Quebec to Montreal seemed particularly favorable for the use of elevator dredges in its improvement for navigation. In the 160 miles of river between these ports there was an aggregate of more than 63 miles which was deficient in depth. Nor were these shoal places comparatively narrow bars typical of many great rivers, but they often consisted of long stretches of shallow water in which the material was largely composed of clay varying in consistency from a rather soft condition to a shale; for example, the widening of the river which forms Lake St. Peter had a natural channel depth of only 10 ft. through its length of about 20 miles. To meet the commercial requirements it was necessary to provide an artificial channel of a minimum depth of 30 ft. and a width of 450 ft. in straight portions with an increase of about 60 percent in width in the curved parts. For this extensive work the Canadian government constructed a half dozen elevator dredges of large capacity whose effectiveness in this especial case has been noteworthy. These six dredges annually remove from 1,500,000 to 2,500,000 cu. yd. during the working season, the river being closed by ice for an average of five months each year. In the soft clay of the bed of Lake St. Peter, elevator dredges have repeatedly approached 500 cu. yd. per hour in their capacity. A unique fact connected with this improvement by dredging is its permanence. A combination of circumstances, which is most fortunate and rare, results in this condition. The principal causes are the stability of the material composing the bed and banks at the existing velocities of the river, the relatively slight range in volume of discharge, and the comparative freedom from silt and sediment carried by the river, the two latter conditions being due to the controlling influence of the Great Lakes from which it flows. The dredging of this channel was begun in 1832, and practically all of the excavation accomplished during each period since that date has constituted a fixed gain in depth. "Surveys and soundings made sixty years ago correspond closely with those of the present day. The work, therefore, is permanent."[1]

72. Hydraulic Dredges.—The commercial interests of the St. Lawrence River required a more rapid enlargement of the channel, especially at Lake St. Peter, where about 15,000,000 cu. yd. still remained to be excavated, and therefore a hydraulic dredge was built in 1901 at a cost of $163,800, and proved perfectly capable of operating on the soft blue clay of the bed of Lake St. Peter at a rate several times

[1] Transactions, Canadian Society of Civil Engineers, Vol. 18, p. 141.

the capacity of an elevator dredge. The requirements of this dredge involved several novel features, such as a wide cut (up to 700 ft., while the usual width is not more than 30 ft. for hydraulic dredges) which necessitated a lateral feed instead of the usual forward feed, and a mechanical cutter capable of loosening the clay rapidly as the dredge was moved laterally across the channel and feeding it without clogging into the suction pipe with the necessary, but relatively small, proportion of water required for this purpose, and also for facilitating its passage through the pipe line to the point of discharge, all under the impulse of the great centrifugal pump through which it passes.

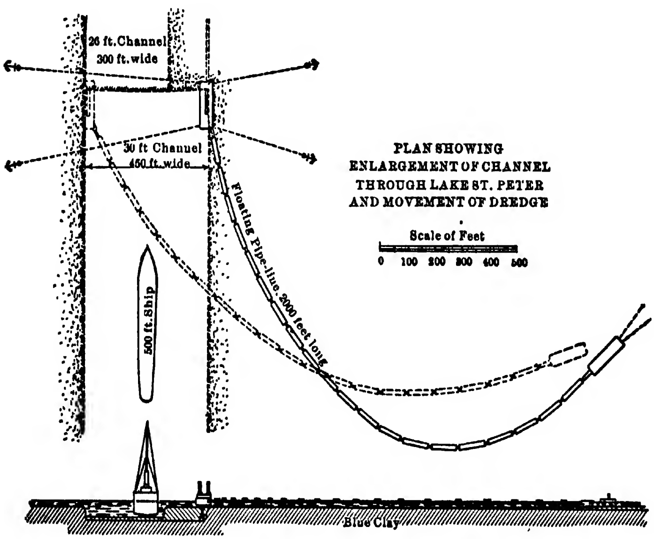


FIG. 79.—Control of a lateral-feeding dredge.

The cutter is of a design especially adapted to the particular material in which it works, 9½ ft. in diameter and 9 ft. long, weighing 10 tons, having rotary steel blades of ample clearance to prevent clogging. The loosened clay and water pass from the rotary cutter into the 40-in. suction pipe, which extends downward through a well in the center of the dredge, through which it is drawn to the 1200-horse-power centrifugal pump having a cast-steel runner of the enclosed type and whose passages are large to facilitate the easy progress of the excavated material. Thence the clay passes into the 36-in. discharge pipe connecting with the pipe line of the same diameter, through

which it is forced to the dump. The rotary cutter, therefore, accomplishes the excavation, and all the hydraulic parts are for the direct purpose of transporting the excavated material from this position to its proper place of deposit. Fig. 79 indicates the method of operation of this hydraulic dredge, which is here shown at the extreme right of the channel in full outline, or at the left in broken outline; and is held in longitudinal position by the head lines shown, running to a substantial anchorage up-stream, and also by a stern anchor line, not indicated, controlled by a special mechanism which automatically keeps a constant tension in this line and so steadies the dredge. There are also seen four breast lines extending, two from each side, to substantial anchors; it is by means of the control offered by these side cables that the lateral movement is given the dredge as it is operated from side to side of the channel. When a steamship must pass, the dredge is placed at the side of the channel nearest the pipe-line discharge and the breast lines to the opposite side are slackened to rest on the bottom. The nominal capacity of this dredge was planned for a working rate of 2000 cu. yd. per hour. In actual service its record capacity has been 2600 cu. yd. per hour for several hours, 757,100 cu. yd. of blue clay in one month of twenty-six working days of 552 working hours, from a depth of 35 ft. and delivered 2000 ft. away, and 2,671,750 cu. yd., scow measurement, in one season of 126 working days in 1903.

Hydraulic dredges (often called suction dredges, and sometimes pump or sand-pump dredges) have had a very great development during the last twenty years, and especially since the beginning of the present century, in response to the imperative demand for an excavating machine of great power, capacity, and relative economy in removing the huge deposits of non-tenacious, easily excavated sediment particularly troublesome in the alluvial portions of rivers. Practically all the great commercial nations have shared in the movement of building machines of this kind. Their use, in such countries as Great Britain, France and Germany, whose waterways are all of moderate size, has been confined mainly to the lower stretches of the rivers and the bars at their mouths; while other countries have largely adopted this type of dredge as best fitted for dealing with conditions on extended portions of their greatest rivers.

Hydraulic dredges for the maintenance of navigable channels at low water in inland waterways have attained their greatest development on the lower Mississippi River. The contract for the first

experimental dredge of this type constructed for operations on this river was let in 1892, experiments and consequent changes in certain of its parts followed its installation, and its first actual employment in aid of navigation occurred in 1894. Since then nine dredges of this type have been constructed, continued improvement and increase of efficiency resulting from the experience gained in their use.

The general features of such a dredge appear on Plate IV[1] (facing p.284), showing a longitudinal section, a plan and an end elevation of one of the latest self-propelling hydraulic dredges built by the Mississippi River Commission. Its hull of steel is 210 ft. long, 44 ft. beam, 8½ ft. molded depth, its draft is 5 ft., its nominal capacity is 2000 cu. yd. per hour and its cost was about $242,000. Describing more in detail its dredging mechanism, the suction head is shown at "*A*," extending downward and forward through the well or opening in the forepart of the hull, this well being 35 ft. long, 33 ft. wide at the forward end and 22 ft. wide in its narrowest after-part. The suction head is of similar outline, but slightly narrower; at its fixed end at the forward bulkhead hinged, telescopic, radial joints permit the necessary adjustability of the outer end to the dredging depth desired; the maximum depth provided for in this dredge is 20 ft. The suction head is 33 ft. long from center of hinge pin to the lip, with its mouthpiece, "*B*," 32 ft. wide and 15 in. high, outside dimensions. It is designed for dredging either up-stream or down-stream and therefore the mouthpiece has both an up-stream and a down-stream mouth. The clear height of each mouth through which the sediment is drawn is 10 in., and both are screened by vertical pipes, bolts and rods so spaced as to leave rectangular openings to prevent the drawing in of objectionable débris. There are heavy strengthening angles riveted on the lower face of the bottom plate of the mouthpiece at both its forward and after edge, through the outstanding legs of which there are openings to accommodate the thirty-six 3-in. jet nozzles, half of them pointing forward and half in the opposite direction. Pipes vertically through the mouthpiece connect these nozzles with the pressure chambers built on top of the mouthpiece. A 20-in. pipe connects these pressure chambers with the 20-in. centrifugal jet pump, situated at the forward end of the engine room, as indicated on the plan, its capacity being 8000 gal. of water per minute delivered at a pressure of 20 lb. per square inch. If the dredge is working up-stream the eighteen forward jets are operating

[1] Adapted from paper No. 6, Sixth Communication to the Tenth International Navigation Congress.

to loosen the sand or mud which is then easily drawn into the open forward mouth of the mouthpiece, while the down-stream mouth and jets are closed; but when dredging in a down-stream direction the forward mouth is closed by the four controlling flap valves and the after mouth is opened by the same control, the jets being correspondingly manipulated.

From the mouthpiece two throat-pieces extend backward in the suction head, each being about 16 ft. wide where they join the mouthpiece, and rapidly contracting by gradual curves, as shown on the plan, to a width of 24 in. to join the two suction pipes which are 2 ft. square and well tied together by a system of lattice bracing. Leaving these square suction pipes of the suction head at its hinged end, the excavated material passes directly through the telescopic joint into the two $25\frac{1}{2}$-in. suction pipes ("*C*") in the hold, connecting with the main dredging pump situated in the center of the forward part of the engine room. These circular suction pipes join this double-suction centrifugal dredging pump at both sides at its horizontal axis, and the water carrying the excavated material is here drawn into the pump, passing through it and thence is forced out through the discharge pipe leaving the centrifugal pump at "*D*." The suction ports of this main pump are each 45 in. in diameter, and all its connections and parts are designed to secure the most free passage possible for the dredged material passing through it; this requirement is especially considered in the design of the interior parts of the pump. The pump runner (as indicated on the longitudinal section) has six blades, three of which are arms of the cast-steel spider and the other three are set midway between those arms; the diameter of this enclosed pump runner is 90 in., the width of the blades is 25 in., and its peripheral velocity is 55 ft. per second.

The discharge pipe ("*D*"–"*E*") runs aft from the main pump through the center of the hold to the stern of the boat. It is 36 in. in diameter and extends through the transom, terminating $4\frac{1}{2}$ ft. outside the stern in a cast-iron ball ("*E*") to permit the necessary jointed connection with the following pipe line.

The excavated material thence is forced by the pressure of the centrifugal pump into the pontoon pipe line from the dredge to its place of discharge. The exterior pipe line provided for this dredge consists of five sections of pipe, each 3 ft. in diameter and 100 ft. long, fitted with ball and socket joints at their ends to secure the necessary flexibility of the line; an idea of the substantial character of these connections may be gained from the necessary dimensions

of the cast-iron balls, which are 5 ft. in diameter and 46 in. in length. The end of the last length of pipe has a baffle plate attached, instead of the usual ball. This is set about 4 ft. from the end of the pipe and is so mounted that its position can be readily adjusted in such a way that the reaction of the discharge against it, when set at the proper angle, is made serviceable in deflecting the pipe line to the situation desired. In addition to the five main sections of this discharge line there is a junction pipe of the same diameter and 8 ft. 4 in. in length, fitted with similar ball and socket joints, necessary for connecting the pontoon pipe line with the after end of the discharge pipe in the hold of the dredge, at "*E*." Sheer legs and tackle ("*G*") are provided at the stern for the purpose of lifting or supporting this junction pipe. For carrying the pipe line at the surface of the river ten pontoons ("*F*") are required. The bottoms of the pontoons are frustums of cones in shape, their decks are 28 ft. in horizontal diameter, their maximum depth is 63 in. and they are well trussed and strengthened for their service. A semicircular trough of 19½-in. radius extends fore and aft in the deck of each pontoon, in which the discharge pipe rests, being held in position by strap bolts and iron bands. Each 100-ft. length of pipe is carried by two pontoons, one near each end, the one next the dredge being shown at "*F*." The displacement of the pontoons is such that they support the pipes entirely above the water surface in order to facilitate operations, such as coupling them together, as well as to render them more responsive to the reaction at the baffle plate to secure the desired position for the pipe line.

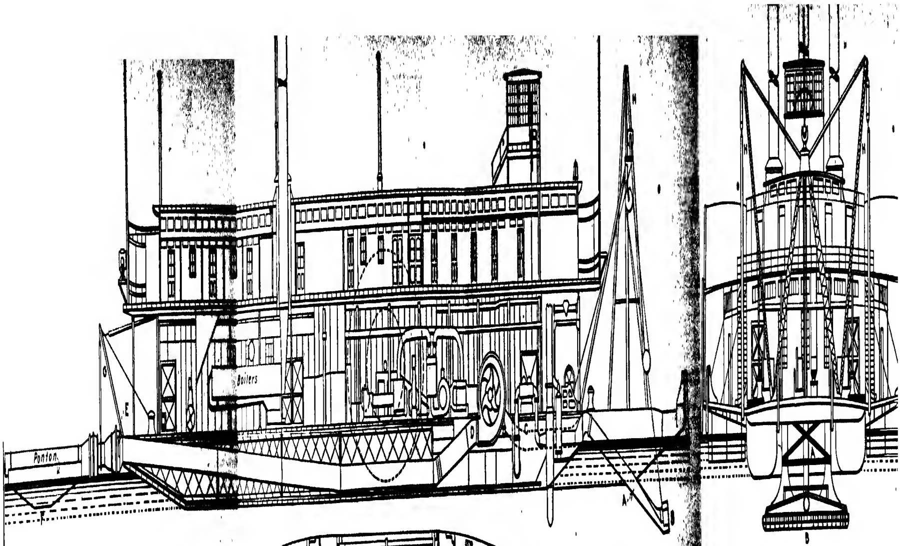
As there is a wearing away of metal on the interior of the pipes and machinery through which the dredged material passes, especially rapid if gravel predominates, and particularly serious at angles and short bends and on rotating parts, sharp angles and bends are avoided as much as possible, parts particularly exposed are made as resistant to the abrasive action as is practicable, and those parts thus affected are so designed that they may be removed when worn and new parts be readily substituted. Even in the straight pipes the wear at the bottom is much greater than elsewhere and it is so serious that the discharge pipes are so arranged that they may be rotated through a portion of their circumference, and thus bring a thicker part of the shell to the bottom when it becomes necessary. When dredging in gravel the pump runner and linings of the casing were so worn in two months' service as to require replacement, and the discharge pipe was cut through.

73. Typical Procedure in Opening a Channel through a Bar.—When a bar requires deepening the typical method of operation consists in steaming to its up-stream edge and there dropping the spud (which is shown on the plan, 12 ft. aft of the end of the suction well) so as to hold the dredge immovable while two hydraulic piles are sunk by water jet 50 ft. or more apart and at one side of (and just above) the upper end of the proposed cut. These piles, of which the dredge has twenty, are 11 in. in external diameter and 35 ft. long; the lower tube, 20 ft. in length, being $\frac{3}{4}$ in. thick and the upper 15 ft. of tube being $\frac{1}{2}$-in. thick, the two sections being joined by heavy cast-steel flanges. The top end of each pile is closed by a steel cap which has a lifting eye cast on it, and each pile has near its upper end a hole tapped with a $2\frac{1}{2}$-in. pipe thread for receiving the pipe transmitting the hydraulic pressure used in sinking the pile to a depth of perhaps 15 ft. into the sand. The sheer legs and tackle for handling these hydraulic piles are shown at "*H*," with a pile suspended from each ready for sinking. After the two hydraulic piles are placed, the hauling cables (1 in. in diameter and 1600 ft. long) are attached to them, thence passing through the fair leads, with sheaves 30 in. in diameter, shown at the extreme right and left edges of the very front of the bow on the plan; these wire ropes pass around the hauling winches as indicated. The spud is then raised and the dredge is dropped down-stream by slackening on the wire rope hauling cables, either dredging as it goes or else dropping to the down-stream edge of the bar before operations begin and then dredging up-stream as the boat is hauled ahead by winding the hauling cables about the winches. Just before dredging begins the suction head ("*A*") is lowered to the desired depth and held by means of the hoisting frame and tackle mounted above it and controlled by the hoisting winch which is shown on the plan just aft of the suction well, and the mouthpiece and set of water-jet agitators corresponding to the direction of dredging are put in operation. The rate of movement of the dredge when excavating varies from about 1 to 8 ft. per minute, depending upon the material and the depth dredged; its average capacity for a recent full working season was about 1500 cu. yd. of sediment per hour when working at depths ranging from 8 to 13 ft.; and, while the percentage of sand carried through the discharge pipe varies greatly, perhaps 10 to 20 percent is a fair representative figure where the mean velocity of discharge is 15 or 20 ft. per second, although the working conditions vary so greatly that the values mentioned must be considered as only illustrative. Side lines for

holding the dredge to the desired direction of the proposed channel are not usually necessary, as the current of the river generally effects this control satisfactorily, and crossing the hauling cables aids control in this respect, if needed. When a cut is made through the bar the head piles are shifted to control the course of the dredge while making the parallel cut alongside the first, and this process is repeated until the channel has been dredged to the necessary width.

The efficiency of dredging a channel through a "crossing" or bar seems to depend upon several factors. Down-stream dredging is often favored for the reasons that the material is thus entirely removed, while up-stream dredging leaves some of the material behind in the cut because of the failure of the suction head to draw into itself all of the sediment thrown into suspension; that a greater rate of progress is secured through confidence in escaping the danger of breaking the hauling apparatus because of the diminished strain on it, permitting a more rapid feed; and that it facilitates the formation of a strong current through the cuts as made, and so lessens the amount of actual dredging required by the erosive action thus induced. However, in general, there is no universal advantage of either procedure over the other. It seems often desirable to excavate even deeper than navigation requires, and to widen the upper end of a dredged cut, so as to encourage the flow of the stream to seek the newly formed channel, and thus to secure its aid in keeping the cut open and even to enlarge it. There are also advocates of a greater depth of dredging at the lower end, however slight the practicable increase may be, for the purpose of conserving the energy of the current as much as possible and so lessening the danger of the silting up of the dredged channel at its lower end. The line of the excavation can generally be kept more nearly straight, especially at its lower end, if the first cut is made at the lower (down-stream) side of the prospective channel.

However, the effectiveness of a dredged channel depends principally upon the choice of the location of the cut with reference to both its direction and its position. Experience proves what theory plainly indicates, that if either factor of the location be carelessly assumed the result is very probably an artificial channel which rapidly fills again with sediment, due to the fact that the river current at the place has a direction crossing that of such a channel, or else is too weak to be of service. The truth is that a dredged cut not only must avoid the adverse influence of possible current effects, but it can, by a proper location, secure the active assistance of the



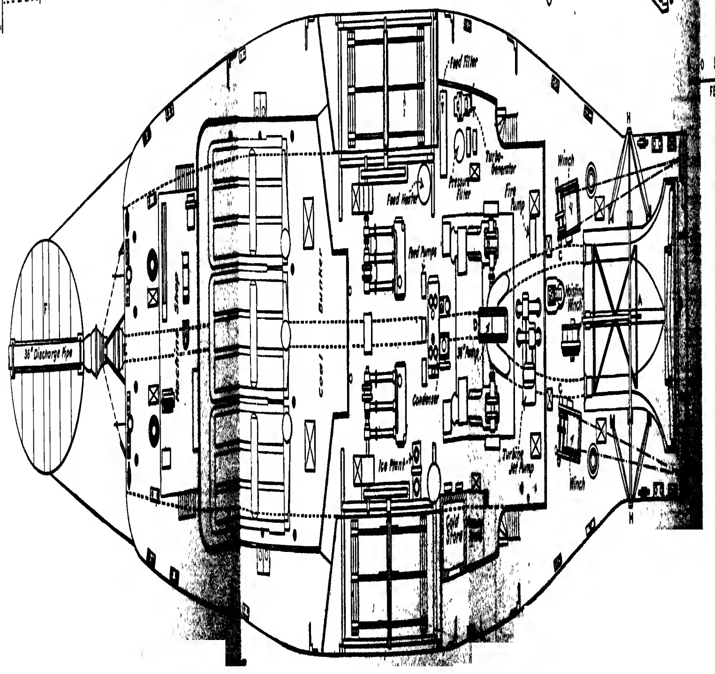


PLATE IV.
The self-propelling hydraulic dredge "B. M. Harrod" of the Mississippi River Commission.

river currents to keep open the channel and aid very much in enlarging it by thus obtaining a scouring velocity which is often of a decided advantage. To secure this desirable location it is necessary that the direction of the proposed channel should coincide with that of the natural river across the bar, and that its position should be that at which the maximum energy of the stream is concentrated at its low stage; experience proves that a channel dredged in the Mississippi River will rapidly fill if either of these conditions is not fulfilled. Both these requisites are the same as those which would guide the location of permanent contraction works if they were to be, instead, built at the place to secure a permanent improvement.

The practical study of determining the location which will minimize conflicting influences and secure the maximum aid of the natural agencies available is mainly dependent upon a hydrographic survey of the crossing made shortly before the dredging is to begin. Such a survey will extend considerably into the lower end of the pool above and the upper end of the pool below, which are to be connected by the proposed work. If these pool ends point toward each other the procedure is evident and unmistakable: the two ends are to be connected directly by dredging, but if, as is usually the case in such great rivers as are involved in this discussion, the ends of the pools do not lie on the same axis and often overlap, as in Fig. 80 (p. 286),[1] the indications are uncertain. To make the cut on the shortest section, or even where the amount of material to be removed is a minimum, is usually a mistake as it generally does not coincide with the place of maximum direct current effect; furthermore such a procedure might involve such sharp turns at each end of the cut that its navigation would be difficult, an objection that usually disappears when the cut is made on the line of maximum current energy. In such cases in addition to the contoured hydrographic maps which by comparison give the principal information available, it is very desirable that their indications be supplemented by additional knowledge of current conditions and tendencies through a general experience of the regimen of the river and an especial familiarity with the characteristic nature and proclivities of each particular crossing, supplemented by further current observations at the time so that the maximum assistance of the stream itself may be definitely secured. It is believed that no better auxiliary method of investigation is generally available than to make use of submerged floats for this purpose; a procedure which

[1] This, and the five following illustrations are reproduced from the Annual Report of the War Department, 1905, Vol. 8, p. 150

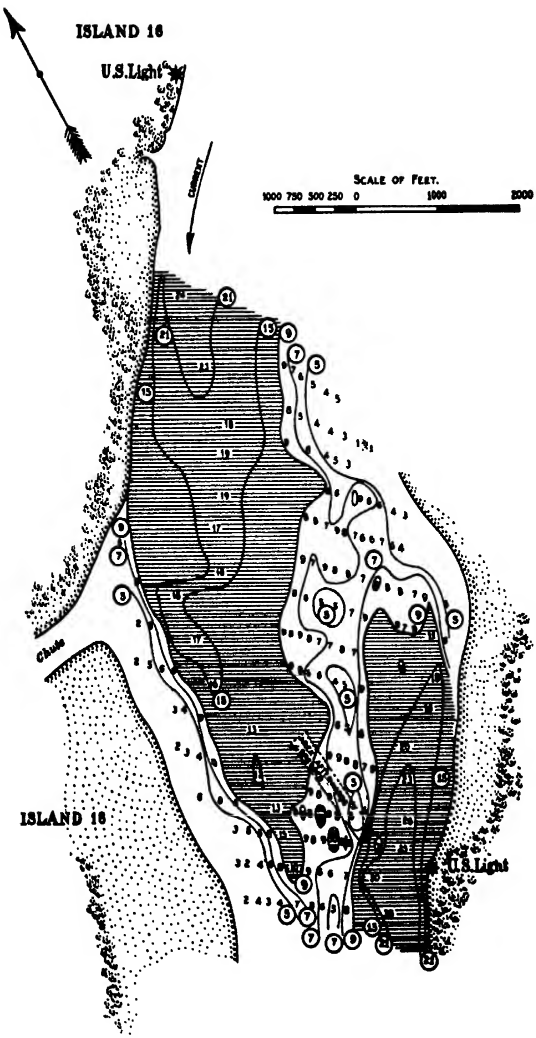


FIG. 80.—Crossing at the foot of Island 16, before dredging.

is too often slighted, but which is inexpensive, quickly accomplished, and designed to directly disclose that which all other investigations

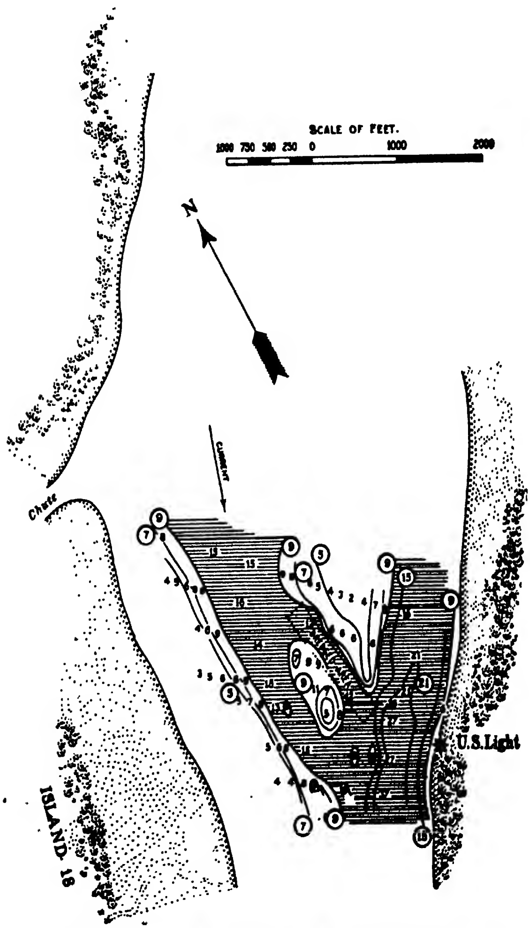


Fig. 81.—Crossing at the foot of Island 16, after dredging.

only indirectly indicate. A popular but crude illustration of the significance that should attach to careful float experiments is given by the fact that experienced river pilots at doubtful crossings give

heed to the course of a floating log that may happen to be passing a bar, it being known by the name of "a good pilot."

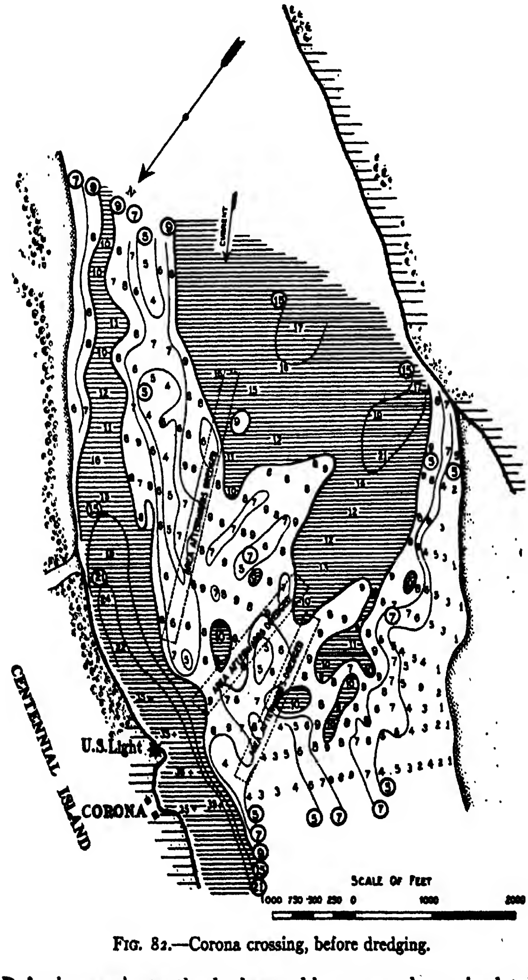


FIG. 82.—Corona crossing, before dredging.

Referring again to the hydrographic survey shown in detail in Fig. 80 (p.286), the location of the proposed cut through the crossing is indicated on it. Dredging immediately followed and the condition

of the "Crossing at the Foot of Island 16," a week afterward, is shown in Fig. 81 (p.287) on which is indicated a navigable depth in the

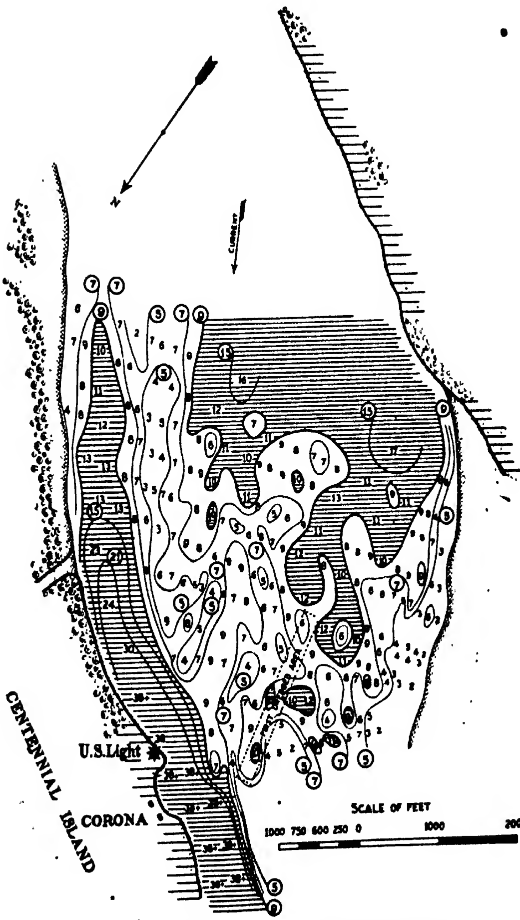


FIG. 83.—Corona crossing, after dredging the first cut.

dredged channel of 12 ft. at m. l. w. The statement that this cut "rapidly improved and remained in excellent condition throughout the season" shows that the location of it was so chosen as to make

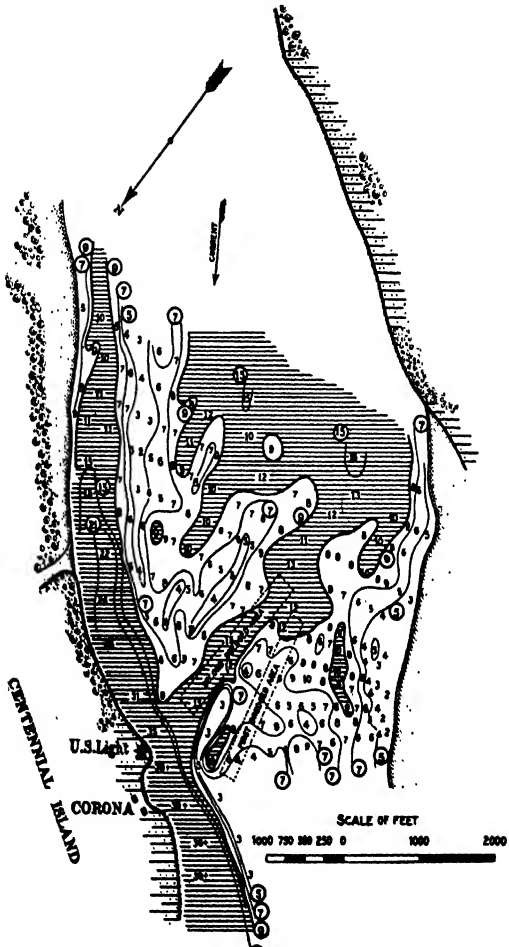


FIG. 84.—Corona crossing, after dredging the second cut.

available much of the energy of the current to aid in excavation not only to keep the channel open but even to enlarge it, thus attaining a considerable measure of a directing or training of the current to accomplish the desired results. However, a comparison of the two

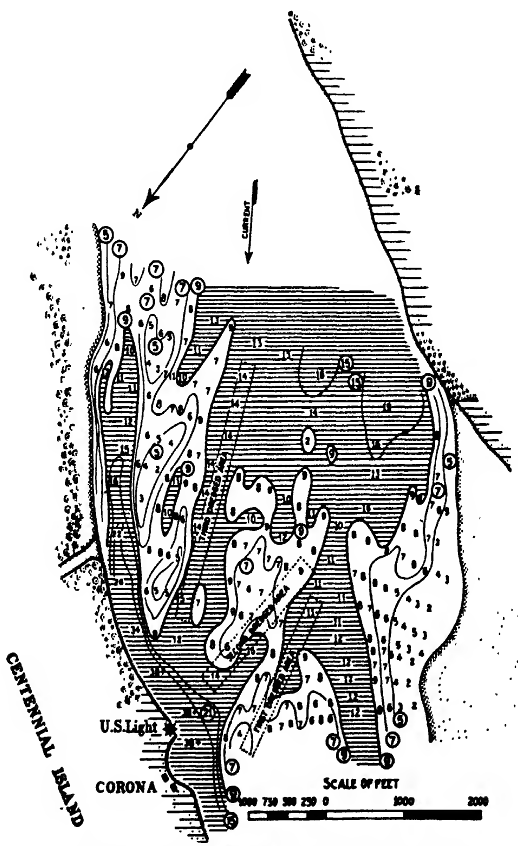


FIG. 85.—Corona crossing, after dredging the third cut.

figures and especially a study of the latter one indicates a marked tendency of the current to gradually displace the position of the channel laterally down-stream, or in this case nearer to the right bank of the river. This shows that a better choice of location for the

dredging would have been farther to the left because the later developments reveal the fact that here the current would have had a still greater influence. Numerous similar experiences have, in brief, developed the general rule that it is desirable to locate the dredged cut as far down-stream as is practicable without increasing too much the amount of excavation involved; a rule which has, however, occasional exceptions, as indicated by the following maps of Corona Crossing. Fig. 82 (p.288) represents the condition in the region of this bar before any dredging was undertaken. Early in September the lowest of the three cuts was dredged, but with poor success as revealed by the survey of three days later in which the channel indicated in Fig. 83 (p.289) is tortuous and not well defined and is hardly 9 ft. in depth, though it lies in the vicinity of the dredged cut. Accordingly, seven days were taken in the middle of the month to open a channel about 500 ft. up-stream from the position of the first one, as shown in Fig. 84 (p.290) which was plotted from a hydrographic survey made the day after the dredging was finished. This location proved successful as a well-defined channel of ample depth was secured, which remained open for at least a month. However, the river pilots objected to the difficulty of navigating this indirect channel and when further dredging became necessary, due to the low October stage, the third cut was opened about a quarter of a mile further up-stream. As indicated in Fig. 85 (p.291), which shows conditions on the day after the dredging was finished, this last channel was both wider and deeper than either of the others, as well as more direct and easily navigated; it also maintained itself in position and ample depth through the rest of the season, even at the lowest stage, while both of the two earlier channels became closed. The report intimates that the last location should have been chosen in the first instance, which is a conclusion that seems evident from a study of the four maps of this crossing, especially if the direction and position of the arrows placed to represent the maximum strength of current were known from the beginning, as this essential factor should always be determined before operations are begun. In fact, it seems to be those cases in which the available current energy is directed upon the bar at a point well above the lower end of the upper pool which generally constitute the exception to the rule of opening the cut as far down-stream through the bar as is practicable. In the last representation of Corona Crossing, the approximate axial coincidence of the current in the upper pool, of the successful (final) channel and of the channel below the dredged cut, are both evident and significant.

As the river declines below its mean stage the engineer in charge of dredging secures his preliminary information from personal inspection, reports of pilots, etc., concerning the condition of the forty bars (on the 750 miles of the Mississippi River below the mouth of the Ohio) which may impede navigation, and prepares to begin operations if the decline continues. When the river falls below a stage of about 20 ft. above low water the hydrographic surveys begin; and if the reduction continues they are repeated for comparative purposes, the location of the proposed cut is chosen, and dredging begins while the river is still many feet above extreme low water. The reasons for beginning the excavating thus early are partly to secure the greatest possible assistance of the current, which often becomes less powerful and effective as the stage declines, and whose aid often actually accomplishes a large share of the opening of the needed channel crossing; and partly to permit the completion of the cut by the time the channel shall be needed, as well as to make the dredge available for work at other shoal places as required, thus reducing the number of necessary dredges in the interest of economy of the service. For all these reasons it is the custom to begin dredging at a stage of perhaps 10 ft. above low water; and also to generally dredge to the full depth which the length of the suction head will permit, at whatever stage the work is done. Such dredged channels are marked by shore ranges and buoys so that their position may be definitely indicated to the river pilots.

The assistance of the current in keeping open and enlarging a cut is much greater when the river is slowly falling than when its decline is rapid; and if a temporary rise in stage intervenes, it is usual to find that the channel has been partly filled again with sediment, especially if its axis is not coincident with that of maximum current energy, due to the influence of the same natural forces which formed the bar itself. Both theory and experience indicate that the longitudinal dredging, which is typical of the Mississippi work, secures a far greater amount of assistance from the current of the river than would the method of either lateral or radial operation which is typical of some of the work carried on elsewhere. Of course, this especial advantage of longitudinal dredging disappears when the energy of current is too slight to be important; but in a river like the Mississippi, where the material is not tenacious and the current greatly aids in the opening and maintaining of the channel, its superior effectiveness is unquestioned.

Usually the dredged material is discharged from the pipe line near

one side of the river, well away from the cut, so as to minimize the danger of its being again deposited in the channel; but sometimes this spoil may be made of use, as in the filling of dead pools (the remnants of channels of previous years which are abandoned by the current in the changes of the river bed) whose existence when near the present channel is a detriment to it because of the amount of flow which they abstract; or as in the case of the upper river where the spoil has sometimes been utilized for the body of closing and contracting works. On the contrary, the material excavated by the dredges of the Volga is deposited on the up-stream side of the cut in order to reduce the area at the side of the channel where the water uselessly flows in a shallow sheet, and thus to concentrate a greater volume of flow through the cut, to its advantage; and the up-stream side of the dredged channel is chosen for this purpose because of the observed decided tendency of such cuts to gradually move laterally in a down-stream direction, as already noted, and so the spoil on the opposite side is generally safe from attack in this movement of the channel.

74. The Cost of Dredging Operations.—To discuss the costs of the dredging of rivers at all satisfactorily is an exceedingly difficult matter, and to give definitely reliable figures is impossible. The reasons for this situation are that the costs not only vary greatly with the character of the material excavated, the efficiency of each dredge, and the skill displayed in its operation, but also upon the scope of the items included in the cost charge and on the method used in determining the volume excavated.

As far as the dredges themselves are concerned, it is considered generally true that as machines they are wasteful of power; the useful effects, in proportion to power expended while actually at work, being still the least for the hydraulic type. This fact, of course, refers simply to their mechanical efficiency, and it constitutes only one factor to be considered in connection with their usefulness; the advantages of large capacity, special adaptability and general availability for the work required outweigh the drawback just mentioned.

That the cost varies with the character of the material concerned is necessarily true, because the rate of removing it depends greatly upon such varying characteristics; and often the cut made extends through earth layers of different kinds, the qualities and amounts of each of which are quite diverse. Variation in material not only enormously affects the cost of its removal with any type of dredge, but it also properly influences the choice of a suitable kind; as in the

case of the St. Lawrence River work, where some are of the elevator type, some hydraulic, and some are dipper dredges; or as on the Volga River, where hydraulic dredges are preferred for sand and silt, but the fact that the same machines which are used in channel excavation must also be employed, when not so engaged, in dredging refuge harbors for protecting the river fleets against moving ice in winter has led to the construction of more ladder dredges which are better adapted to the work in the hard clay containing roots, logs and other débris as found in those harbors and sometimes in the channel.

The effectiveness of operations depends greatly upon skillfulness in both the choice of location of the proposed channel of sedimentary rivers as already explained, and in the detailed control of the work of the dredge to secure a maximum output. This last consideration is especially true for hydraulic dredges, whose rate of excavation is so intimately connected with a harmonious operation of rate of feed, control of the jet agitators, and other details in order to secure a maximum percentage of solid matter in the water flowing through the suction and discharge pipes. Probably the great range in unit costs, which follow, is mainly due to these two items; the degree of ability displayed in the control of the dredging and the variability in the materials excavated.

In computing unit costs, different authors and authorities use varying methods of arriving at the total expenditure made for accomplishing the aggregate excavation involved. If such estimates of cost would state just what charges are included, the statements would be much more valuable; but often these are not given. The probable reasons for so great variation include not only the fact that estimates of costs often fail to make the proper charge for capital expenses of all kinds in addition to operating expenditures, but that the working season is short during which the dredges may be needed; while the actual dredging operations only occupy a small fraction of even this short working season because of rising stages of river, changing from one locality to another, and other interruptions of various sorts. Illustrating these conditions are the facts that the Mississippi River dredges are in commission only about four months out of the twelve, the remaining eight months not requiring their operation and so being entirely unproductive except as it is utilized for their general overhauling and repair in order to put them in their best condition for the next season's work; and that their average number of hours at work for a recent season was only a trifle above five hundred, of which about seven-eighths was actually utilized in pumping. Apparently

the more usual method is to include in the costs all the actual expenses incurred in the dredging season, but sometimes with and sometimes without the additional charges for repairs and maintenance through the rest of the year; yet there seem to be cases in which only the bare cost of fuel, wages, etc., while working, was taken, thus even excluding costs of repairs and accessory expenditures while in commission, such as making the necessary hydrographic surveys, expenses when temporarily idle, etc. It is believed that a trur estimate would not only comprise the total yearly expenditure foe the dredges for all purposes, including a proper proportion of office expenses, salaries of the engineering staff, etc., but also a just charge for the capital invested in the plant and for depreciation. In short, all the capital costs and the annual expenditures upon all the dredging fleet should be included in the estimate in order to give a just statement of the actual costs, whether river conditions are such that all or only a part of the fleet is in commission, or whether it is fully employed or not when on the river; for such expense is incident to the requirements of the service of only opening channels when low water demands it, and so is in the nature of an insurance of navigability, and therefore is germane to the question.

In estimating the yardage of material excavated there is a corresponding lack of uniformity. One system makes use of measuring barges into which the material is deposited from the discharge pipe; while this is a very desirable feature of an efficiency test of a dredge in determining its acceptability, it cannot be employed in the regular operation of hydraulic dredges because of the unnecessary added expense it involves. Only where the type of dredge or the conditions controlling the deposit require the hauling away of the spoil in barges is this method available; but when it is practicable, such measurement is accurate and satisfactory. For hydraulic dredges in regular service the quantity of excavation has often been estimated by what is known as the percentage method; that is by pitometer measurement of the velocity, thus determining the volume of discharge from the dredge, and multiplying this by a fraction supposed to represent the proportion of solid matter in the discharge. Inasmuch as this proportion varies from perhaps 5 to 30 percent, and sometimes more, depending upon rate of feed, kind and control of agitators, velocity of discharge, nature of the material and numerous other factors, it will be evident that this method is totally unreliable. The other system ordinarily used is measurement in place, which is based upon the indications of hydrographic surveys made before and after the dredg-

ing is done. From a comparison of the soundings made throughout the area, concerned, the volume of material removed is readily computed. This method is also entirely inaccurate as a measure of the work actually done by the dredge because the eroding action of the current removes an unknown portion of the total material that is gone; this part being small in poorly located cuts, there even being cases where dredges have worked hours at a bar and have found no diminution in its volume because the current has deposited sediment as fast as the dredge removed it; but ordinarily in well-located cuts the current helps greatly, yet to an entirely unknown extent, as indicated by such statements as that only a small portion of the sediment removed actually passed through the dredge, or that the result of the work of a single dredge at a bar resulted in the removal of material greater in total volume than the capacity of the entire fleet of nine dredges for the time involved. While, therefore, there is no practicable way for determining the actual quantity of material removed by a hydraulic dredge, it is believed that the last method given is the most significant because it measures the actual volume displaced in securing a navigable channel by all the agencies involved; but neither this nor any other method of measurement gives an entirely reliable yardage basis for either comparing the efficiency of dredges themselves or for forming a safe estimate of the actual cost of dredging.

TABLE No. 5.—COST OF DREDGING OPERATIONS ON LOWER MISSISSIPPI RIVER

Year	Yardage removed	Cost in cents per cubic yard for			
		Operation	Care and repairs	Miscellaneous	Total
1900–01	1,145,558	7.6	9.6	1.0	18.2
1901–02	1,666,465	7.8	4.6	0.4	12.8
1902–03	813,380	12.2	11.8	1.6	25.6
1903–04	891,098	13.6	13.1	1.3	28.0
1904–05	2,149,734	7.2	6.3	1.0	14.5
1905–06	197,847	34.4	55.3	5.0	94.7
1906–07	297,300	19.8	31.8	2.3	53.9
1907–08	1,151,739	8.7	9.1	1.4	19.2
1908–09	2,167,766	7.9	5.0	2.8	15.7
1909–10	1,260,171	9.3	11.0	1.4	21.7
1910–11	2,471,040	6.3	5.2	4.1	15.6
1911–12	1,160,333	8.9	10.2	1.3	20.4
Average for twelve years	1,281,036	8.3	7.7	2.1	18.1

A summarized statement of costs on the lower Mississippi River for twelve years is given in Table No. 5 (p. 297) as computed from recent "Annual Reports of the Chief of Engineers, U. S. A." The hydraulic dredges operated mainly in sand, mud and gravel at depths usually between 7 and 15 ft. "Cost of Operation" appears to include expenditures for labor, subsistence, fuel and supplies, as well as field repairs for the dredges and auxiliary outfit while actually engaged in the dredging service during the low water season of each year. "Care and Repairs" and "Miscellaneous" expenditures seem to include all the similar expenses for maintenance and repairs of the dredges and auxiliary floating equipment when not engaged in dredging operations, that of the shops and other shore equipment maintained for the care of the fleet, and the costs connected with surveys, gauges and gauging service and all other complementary operations required for the effectiveness of the whole organization. Expenditures for "Permanent Outfit" are not included.

The wide range in yardage removed is due to the great variations of stage of river for different years. For example, the season of 1904–05 was marked as one having a long-continued and very low water, necessitating a large amount of dredging to maintain the channel depth of 9 ft.; while in the year 1905–06 the season was distinguished by comparatively high stages, which made necessary only a small amount of excavation. In order to assure the maintenance of the required channel depth it is necessary to have a complete dredging equipment sufficient at all times to cope with the most adverse condition which may occur. This keeps the general charges at a fairly constant amount, and when the quantity excavated is small the corresponding unit costs are relatively very large. Even the unit cost of operation is also large in such years because of several of the dredges being in commission in case they are needed, although their actual operating time may be small; in fact, dredges are not infrequently sent into the field with full outfit and crew, ready for any needed excavating, and find it unnecessary to remove a yard of material during the season. When the excavation is large the expenditures per cubic yard are, for similar reasons, noticeably small. If the unit cost of operating individual dredges be considered it will be found that the range is also great, often varying from about one-third the average to two or three times the mean cost.

It will be seen that the average total expenditure per cubic yard for the hydraulic dredging on the lower Mississippi River during the twelve-year period is about 18 cents, and the unit cost of operations

while in commission is about 8½ cents. No estimate for capital charge and depreciation is available.

On the Volga River in the season of 1901 the volume of material removed, mostly by elevator dredges, was 4,804,600 cu. yd. The cost of operation was $374,880, and that for repairs, $28,407, or a total of $403,287.[1] This means a unit cost of about $8\frac{4}{10}$ cents per cubic yard, and does not include charges for capital invested in plant and equipment or for renewal, etc. In 1900 the volune of the excavation was little more than half the amount in 1901.

The cost of maintaining a navigable channel in these rivers may be expressed in another way. On the 300 miles of the Volga on which the greater part of the dredging is concentrated, the average annual cost, for several years, per mile of river was $357 for the maintenance of a navigable depth of 6 ft. Similarly the average annual cost for twelve years of maintaining a channel depth of 9 ft. on the 750 miles of the lower Mississippi River between the mouths of the Ohio and the Red Rivers has averaged about $310 per mile.

On the St. Lawrence River the dredging season is considerably longer, and the continuity of the work is notably greater, which tend to reduce the unit costs; but the excavated material is more difficult to handle, and the depths are much greater, both of which facts exert a contrary influence. During the fiscal year ending March 31, 1912, there were three types of dredges at work. The following summary of costs is based on scow measurement with the exception of one of the hydraulic dredges, and the expenditures as given seem to include all items involved except a proper charge for first cost and deterioration.[2] The three hydraulic dredges excavated somewhat more than two and one-half million cubic yards during the season, working in clay, sand, gravel, and stones at maximum depths of 30 and 35 ft.; the unit costs varied from 5 to 11 cents, the average for the season being 8½ cents per cubic yard. The 6 elevator dredges excavated more than one and one-half million cubic yards to the same depths and in the same material except that there was considerable hard-pan and shale also; the unit costs varied from 5½ cents to more than a dollar, and averaged 20 cents per cubic yard. The two dipper dredges also worked in all the materials already mentioned to a usual depth of 30 ft.; their unit costs varied from 11 to nearly 48

[1] Transactions, American Society of Civil Engineers, Vol. 52, p. 225: "A Desirable Method of Dredging Channels through River Bars," by S. P. Maximoff.

[2] Forty-fifth Annual Report of Department of Marine and Fisheries, Dominion of Canada.

cents, and averaged 14 cents per cubic yard for nearly half a million cubic yards excavated. Of the total amount of dredging necessary to deepen the St. Lawrence River ship channel to the recently projected depth of 35 ft., amounting to about 134,500,000 cu. yds., there had been excavated, from 1851 to the year 1912, a total of 78,231,531 cu. yds. of all classes of material varying in composition from soft blue clay and sand to a very hard shale rock; the total cost was $14,524,555, of which nearly 40 percent is for items not usually included in such charges, such as surveys, shops, cost of the dredging plant itself, etc. The gross cost for all the work in all materials encountered during these 60 years therefore amounts to a trifle more than $18\frac{1}{2}$ cents per cubic yard, which should be reduced by a proper unit credit to cover the unreported value of the equipment on hand.

It is believed that the unit costs given above are far more representative than statements that sometimes appear in which the values are given as low as four, three, two, and sometimes even less than one cent per cubic yard. Of course, exceedingly low figures may be obtained on short time tests under unusually favorable conditions; and values such as some of those just given may be attained when operating for weeks if the material is easy of excavation, the dredge in excellent working order and operated continually and expertly, and especially when the current's active aid is included and the costs charged consist only of the actual expenses incurred during the time at work. Such values are interesting as showing what may be occasionally accomplished, but are worthless in considerations involving general average cost data.

75. Conditions Influencing the Effectiveness of Dredged Channels. —Regarding the effectiveness of dredging in keeping open the required navigable channel of 6 and 7 ft. on the alluvial portion of the Volga and 9 ft. on the lower Mississippi, it should be said that general success has been attained in both. There is, however, a noticeable difference between the two rivers with respect to the natural maintenance of dredged channels. On the Mississippi the high waters of the spring and early summer obliterate the work of the previous fall to so great an extent that usually no advantage from the operations of a previous low water season is secured for the lessening of the necessary amount of dredging; and even a cut made in the earlier part of a dredging season may be partly or wholly filled by a rising river, requiring a second opening of the channel, although when once made it frequently remains effective through the re-

mainder of the year. Experience on the Volga indicates a considerable degree of permanence. Even a stage of 30 ft. above low water seems often to produce comparatively little effect in the filling of a majority of the cuts previously well opened, unless such a high stage is of long duration. In some channels the depths have been found greater a year after they were dredged than they were when the work was done. Only a small portion of the cuts require renewal every year. These results are largely attributed to the particularly active scouring effect of the current occurring when the river is frozen over, a condition which exists for about one-third of the year on this river in the latitude of Labrador, and which is entirely absent on the lower Mississippi, 1500 miles nearer the equator. Another reason is found in the fact that the Volga has in general a more stable regimen than has the Mississippi, which is evident in comparing the rate of erosion of the alluvial banks and the activity of the bar-forming proclivities of the two streams. Both rivers are alike in the general tendency for dredged channels well located to improve with a slowly falling stage and to deteriorate in navigability at high water periods.

It may be well to more definitely state the controlling characteristics of these alluvial rivers for the particular purpose of showing their similarity in those features which have led to dredging as the predominant method of maintaining their required navigable depths, as well as to disclose some contrasts which help to explain differences in the character of operations. It is generally considered that dredging should be the particular method of improvement in rivers of notably unstable regimen, in streams of such great size that the time required for construction and the cost of contracting works would be excessive, and where there is such moderate slope that the effect of dredging will not seriously affect its general low water characteristics. The relative rapidity of descent of rivers of ordinary regimen as contrasted with great alluvial rivers may be judged from the statement that an elevation of one hundred meters is found on the Rhone at a distance of 140 miles from its mouth; on the Rhine, 390 miles; and on the Savannah, 270 miles; while on the Mississippi the distance is 1130 miles, and on the Volga about 1300 miles.

On the lower Mississippi River the extreme range in stage is about 55 ft. in its upper part, finally reducing to zero at its mouth. Its low water discharge at the mouth of the Ohio River is about 65,000 cu. ft. per second, and its flood volume is over 2,000,000 cu. ft. per second; both values are increased somewhat by the added volume of the lower tributaries. The low water slope in this distance of 750

miles averages about 4.2 in. per mile above the mouth of the Red River, and this point, at low water, is only 3 ft. above the mean level of the Gulf of Mexico, although the distance by river is 295 miles. The volume of silt transported by this river is enormous, amounting to more than 400,000,000 tons per year.

On the 1060 miles of the Volga below the mouth of the Kama River, the extreme high water stage is about 40 ft. The low water discharge is about 97,000 and the flood volume somewhat more than 1,400,000 cu. ft. per second. The low water slope averages nearly 3.2 in. per mile. The burden of silt appears to be much less than that of the Mississippi. This last feature, and the fact that the range in volume of the Volga is only about half that of the Mississippi, are significant in connection with the greater permanence of dredged cuts on the former river.

In the discussion of dredging on the Mississippi, prominence has been given to the necessity for an advantageous choice of location for the position of the cut if satisfactory results are obtained. The same requirement pertains equally to similar work anywhere in order to secure, in addition to the mere excavation performed by the dredges themselves, that active assistance of the current which is available as a contributory factor of great potentiality. Not only is the energy of the current thus serviceable in the opening of a cut across a shoal place, but its influence in preserving the permanence of the channel is pronounced. This results from the dredged cut acting to a considerable extent as a channel which has its own immediate banks and bed with a resulting cross-section and slope that produce its particular velocities and other characteristics more or less independent of the sheet of water flowing at each side, much as a river which has overflowed its banks has still its channel characteristics which are largely independent of the fact that there is a vast expanse of water flowing over the lowlands beyond its banks; the difference would seem to be one of degree rather than of kind. This tendency of the water to actively seek and maintain such a dredged channel both conforms to the theoretical principles of hydraulics and is an observed fact of experience in such operations; it is known as the "draw of the waters" (l'appel des eaux).

The principles of river hydraulics contributing to this effect are, briefly, that when a shoal separates two deep pools, the volume of flow spreads over a wide section with numerous embryonic channels struggling unsuccessfully to develop themselves over various portions of the bar; there results a great dissipation of energy, which

is therefore unavailable for channel-forming purposes. When a cut is opened the resistance to flow in it is much less than elsewhere over the shoal, with the corresponding effect of a relatively large volume and velocity of flow. If its location be properly chosen it not only exerts a decided inducement for the current of the pool above to extend itself through the cut because of its regularizing influence on the currents and the reduction in the circulation of silt, but the relatively considerable local changes in the regimen of the stream produced at the place are found to attract the waters (beginning to wander at the lower end of the upper pool because of the obstructing character of the shoal) increasingly toward the upper end of the cut as its influence becomes more definite; hence the designation, the draw of the waters. As the local temperament of the stream thus changes there is also manifested a favorable tendency toward the continued shoaling of the bar on both sides of this artificial channel, due to the reduced volume and velocity of flow outside the cut inducing increased sedimentation there; and as the crest of the bar thus slowly builds up, the effect is to throw the volume and strength of the river still more into the dredged channel. All these gradually changing conditions of the movement of the waters at the shoal tend toward a permanently improved channel across the bar, and consequently this method is sometimes called a method of regulation. Advocates of the utility of the draw of the waters are united on the desirability of a dredged channel of minimum width but as deep as possible to produce a maximum effect in its immediate influence and in securing a greater permanency. It is also true that a repetition of operations in such a cut is necessary for a considerable time in order to maintain this situation until the changed conditions may become permanent, but these successive dredgings become less and less in amount under favoring conditions.

Whether or not skillfully planned dredged channels will become permanent depends mainly upon the regimen of the river. In occasional cases where this is favorable and the material of the bed is tenacious beyond the power of the current to dislodge it, as on most of the St. Lawrence River shallows, the results are comparatively permanent. The same result is true to a degree where the bar-forming proclivities of the stream are so relatively feeble and vacillating that they may be counteracted by operations which will cause the induced effects of the draw of the waters to the dredged channel to preponderate, as illustrated by experiences on the Volga. But usually, when the fluctuations in stage and volume of flow are great

and especially when the burden of silt and rolling sediment of the river is heavy, as on the lower Mississippi, sedimentation at the bars is sure to largely or wholly overpower the temporarily induced influence of the draw of the waters, and thus to obliterate its effects.

76. Rock Breakers.—When rock of such hardness as to seriously impede the dredging operations is encountered in channel work, as in the permanent deepening of the St. Lawrence River, the most advantageous way of breaking it up so that dredges can handle it is often found to be the use of a rock breaker. This apparatus consists essentially of a rock cutter or ram which is usually 15 to 20 in. in diameter and 30 to 50 ft. long, depending upon the depth of the rock ledge, and it weighs from 15 to 25 tons. It is made of forged steel and the lower end of the ram is fitted with a socket to receive the striking point made of especially hardened steel to resist the impact of the blow and the wear as effectively as possible. This point is shaped like that of an armor-piercing projectile and its hardness is made to increase toward the center so that it will preserve its general shape as it wears. The ram is lifted by a wire rope which engages its upper end and, passing over a sheave at the top of the supporting frame, extends to the hoisting winch by which its operation is controlled. A guide for the ram is fitted with heavy springs to minimize the lateral shock resulting from the blows. The ram generally works through a vertical well in the center of a barge on the deck of which all the operating machinery is mounted. The crushing of the rock is accomplished by raising the ram 6 or 8 ft. and then allowing it to fall; the impact of the blows progressively shatters the ledges or layers. In the St. Lawrence hard shale it required an average of five blows to penetrate 3 ft. when successive positions of the ram were spaced 5 ft. apart. The broken rock was found to be of a convenient size for the dredges to handle, the fragments averaging about one-fourth of a cubic foot in volume, and their output was increased about 75 percent by the preliminary shattering of the rock, with notably less strain and consequent breakage and repairs of the dredges. The wire hoisting rope and the striking point wear especially fast because of their severe usage, the life of the former often not exceeding 500 working hours, and of the latter perhaps twice as long. The skill with which a rock breaker is operated has a very great influence, not only on the rate of wear, but also on the effectiveness of its work. Where large quantities of rock are to be removed rock breakers have been often found much superior to the older and more usual system of drilling and blasting. Such advan-

tages are stated to be a greater freedom from accidents, operations where it would be injudicious to use explosives, the breaking of rock to a more convenient size so as to noticeably expedite its subsequent removal, more rapid work and less cost. In connection with the latter items there is recorded an instance where the average of twenty-seven weeks of work with a rock breaker at a mean depth of 30 ft. was 1177 cu. yd. of rock per week at an average cost of 20 cents per cubic yard for operation and maintenance, or of 30 cents per cubic yard when interest on capital invested, depreciation, renewals and insurance are included. The cost of drilling (by power churn drills) and blasting in the same calcareous sandstone rock was 72 cents per cubic yard and the average rate was 488 cu. yd. per week.[1]

77. The Advantage of Auxiliary Permanent Works.—Generally where dredging constitutes the characteristic method of improving the river channel for navigation, permanent works of regulation are not absent. On the lower Mississippi River bank revetment is being constructed at the caving banks as rapidly as funds are provided, cut-offs are prevented, and side channels are frequently closed to concentrate the low water flow in the main river bed. All these permanent works assist greatly in changing the stream to a more stable regimen and the bank protection reduces the amount of the silt whose presence is the principal contributing cause of the formation of the bars. Therefore, as such works of regulation extend in number and completeness, the stability of the dredged channels should become correspondingly increased. It is probable that in time contracting works may advantageously be added to an extent which the development of the improvements shall indicate to be desirable, and thus reduce the annual outlay for dredging to its economic minimum.

Similar tendencies are perceptible on the lower Volga, though to much less degree because of local conditions and the greater stability of the dredged channels. Here the auxiliary aid of permanent works is now practically confined to the closing of such secondary channels of the river as experience indicates will result in an increased stability of the dredged cuts. Here, again, a certain extension of permanent works is not unlikely as the navigable importance of the river becomes greater and more clearly defined.

Undoubtedly the stability and availability of navigable channels are increased by the aid of permanent works. It is a question of

[1] Engineering News, Vol. 59, p. 676.

expert adaptation, to the regimen of each stream and accompanying local conditions, of that especial method or combination of methods of improvement which will secure the required results at a maximum economy, which must be worked out for each case.

CHAPTER IX

LEVEES

78. The General Characteristics of Notable Systems.—The employment of embankments to protect lowlands from submergence by flood waters has been the practice of civilized nations from times of remote antiquity to the present. Notable among the many examples of their use are those of the Po Valley of northern Italy, that of the Tisza of Hungary, the famous lowlands of Holland and those along the lower Mississippi in this country.

The raising of lines of earthwork to form artificial banks to prevent the high waters of the Po from overflowing its alluvial plains is particularly conspicuous among similar undertakings because it is the only instance in which the history of a great levee system extends through a score of centuries. Before the Romans developed an extensive plan of flood protection in this valley there existed a considerable system of embankments which Pliny attributed to the Etruscans. The Roman works deteriorated greatly during the period of the barbarian invasions of Italy; but reconstruction and extensions were resumed in the middle ages with such activity that the principal features of the present system were outlined by the close of the fifteenth century.

The area now reclaimed from overflow by the levees of the Po amounts to about 2900 square miles as indicated in Fig. 86 (p.308).[1] The river is embanked continuously on both sides from the sea to the vicinity of Cremona, a channel distance of 270 miles; for about 50 miles above that city there are also quite extensive works of flood defense on one side of the river or the other, and in many places on both sides, while occasional ones have been built in the stretch a hundred miles further up-stream. In addition there are levees which follow the lines of the many affluents across the lowlands to the main river; there are also other embankments bordering the various mouths of the Po in the lower 65 miles of its course; and there is a very extensive system of subordinate levees, built near the immediate banks of the river and to a moderate

[1] Annales des Ponts et Chausées, 1860 (2), Plate 187.

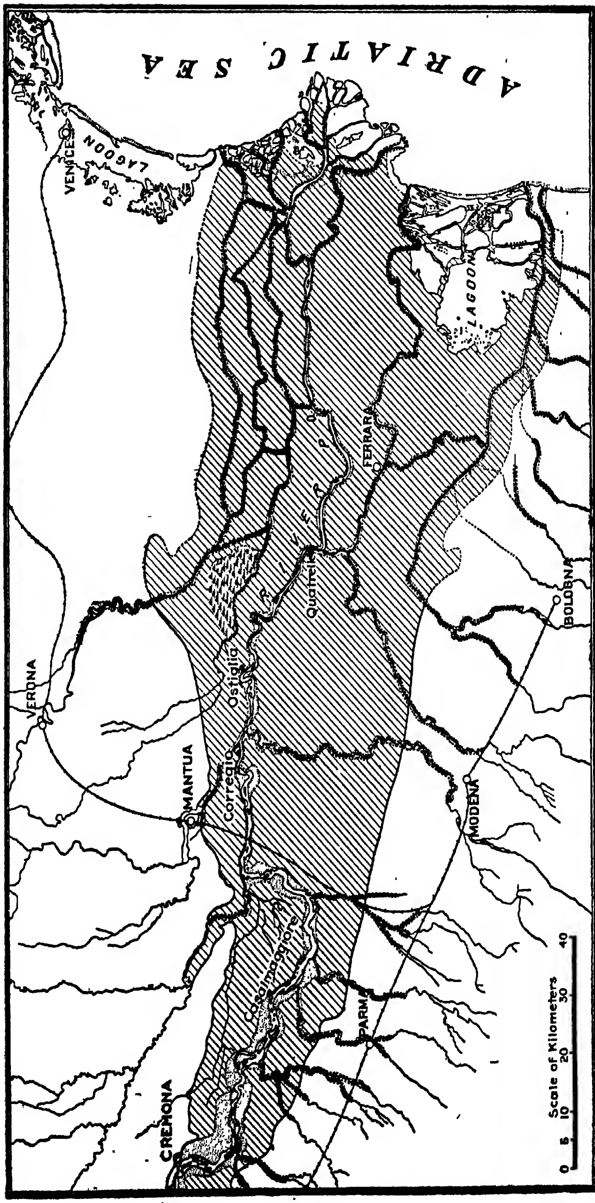


FIG. 86.—The lower valley of the River Po.

height, the purpose of which is to exclude only the ordinary high waters from the fields that are covered by the great floods of occasional occurrence, and whose aggregate length is very great. The surprising variation in the total length of levees of the Po valley, as given by different authorities, undoubtedly arises from including more or less of the several kinds just referred to. But the double line of main embankments along the Po, mentioned at the beginning of this paragraph, is the primary defense of the whole valley; and it constitutes the principal factor of any scientific discussion of the adequacy and effectiveness of levees in that lowland country.

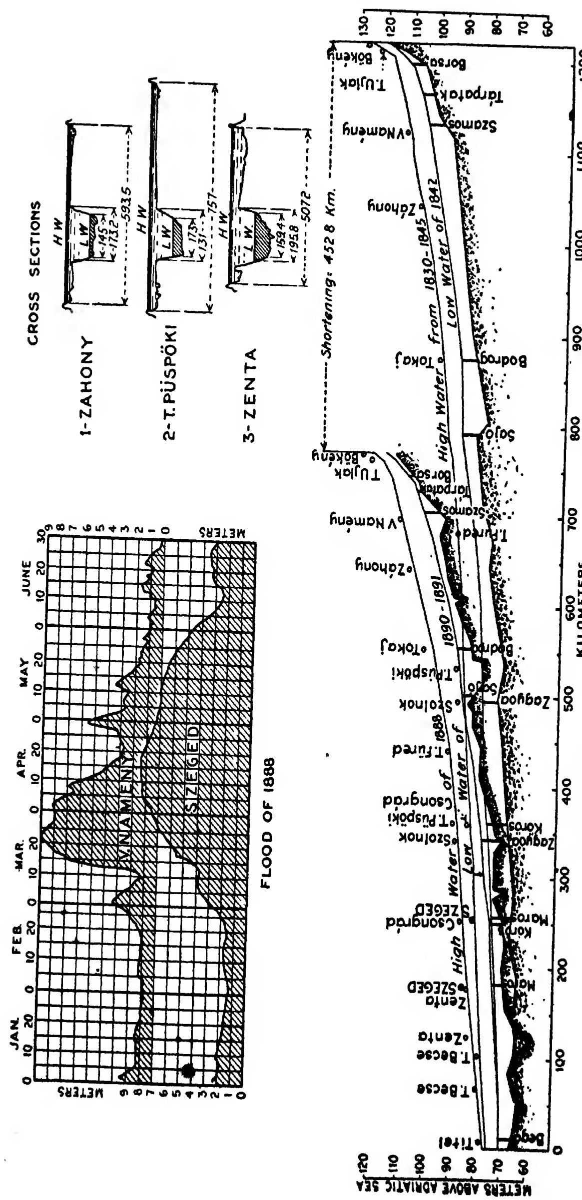
The distance apart of the great embankments on both sides of the main river is exceedingly variable. Referring again to Fig. 86, it is seen that they follow quite closely the windings of the various mouths, and their spacing on each of these streams appears to be fairly regular; the high water channel width in this delta portion, within 65 miles of the sea by river, averaging about one-fourth of a mile and varying from this mean value in extreme cases not more than 30 percent. In the great central portion of the valley from Cremona to Quatrelle, where the Po keeps to its single channel for more than 200 miles, not only do the curvings of the levees fail to even roughly follow those of the river but their distance apart is exceedingly variable. For example, the high water width is only ½ mile in the vicinity of Corregio (100 miles from the sea), while it exceeds 4 miles in the region of Cremona (270 miles). In fact, the general decrease in width of the flood channel in the direction of flow is a characteristic feature of this general system of embankments. The width of several miles between levees in the middle part of the river, where the affluents are most numerous and do most copiously contribute to the volume of flood flow, has a very salutary influence in securing a moderate high water condition throughout that region; and its enormous capacity really has the controlling effect of a storage reservoir at just the place where it will most effectively mitigate the seriousness of floods. But the irregular narrowing in the lower portion, and especially the sudden reductions in width which exist in many places, produce a very considerable increase in height of stage above that which would occur if the spacing of the embankments were less restricted and more regular.

The surface slope of the lower 20 miles of the Po is only about 2 in. per mile; the average for the delta part is 5 in., and that of the remaining distance is about 9 in. per mile. The mini-

mum discharge of the lower river is given as 7500 cu. ft. per second; its average volume, 60,000; and its flood discharge, 250,000 cu. ft. per second. The extreme range in stage seems to vary from about 21 ft. in the wide stretch between Cremona and Casalmaggiore to about 32 ft. where the levees are so much nearer each other in the region of the head of the delta. The general elevation of the valley lands, from Cremona to the vicinity of Ostiglia on the left bank and to the head of the delta on the right side, usually ranges from 12 to 18 ft. above its low water surface. In the remaining distance to the sea they are frequently less than 5 ft. above, although exceeding this small elevation in many places. Fortunately the alluvial valley of the Po exhibits the typical characteristics of rivers of that kind in having gradually built up the adjacent territory, above the general surface of the low plains during the centuries of overflow antedating the construction of levees, by the deposit of sediment from the flood waters as they lost their high velocities in spreading over the valley lands. The natural embankments thus formed have been raised from a small amount where overflows were infrequent to 15 or 20 ft. above the general level of the least elevated parts of the valley. Consequently the height on the levees reached by the crest of a flood is only a part of the total flood rise, rarely exceeding 50 percent of the extreme range in stage and usually being less. The banks of the Po which are subject to erosion have generally been left without protection works, with the result that levees which may be thus undermined are rebuilt on a new alignment farther from the river bed.

The system of works of flood protection of the Tisza valley, which was inaugurated about the middle of the nineteenth century, is not only important because of the great area reclaimed from destructive inundations, amounting to about 10,000 square miles of valuable lands, but it is unique in the extensive employment of cut-offs to reduce the distance of travel of the flood waters and so to facilitate their discharge into the Danube. The increased velocity of flow, thus obtained, made practicable a considerable reduction in both the length and the sectional dimensions of the levees below those which would have been required if the course of the river had not been so largely rectified. The amount of shortening of the stream thus obtained may be seen in the profile of Fig. 87,[1] which shows the original length of 758 miles reduced by the 112 cuts to the present length of 477 miles. Such a radical shortening of a river is only

[1] From the sixth Brochure of the Eighth International Congress of Navigation.



PROFILES OF THE TISZA, BEFORE AND AFTER RECTIFICATION

FIG. 87.—Characteristic features of the Tisza River.

permissible in one whose velocities are naturally so inconsiderable that they may be increased by the amount due to the proposed rectification without becoming destructive to the stability of its régime, or to the interests of navigation.

The Tisza is probably the only river of its size which was naturally characterized by a remarkably slight low water slope combined with numerous great, winding bends that could be eliminated by cut-offs at an expense which would not be prohibitive. Its rate of fall, as indicated on the profile, between Tisza Ujlak and Tokaj, averaged hardly 5 in. per mile; between Tokaj and Csongrad, 1.6 in.; and below Csongrad, 1.3 in. per mile. Its rectification increased these values to 7.8, 2.9 and 1.7 in. per mile, respectively. The greatest of the slopes, that of the rectified 130 miles of the upper part, is still quite moderate for a river whose low water discharge is there but 2120 cu. ft. per second; and that of the 340 miles from Tokaj to the Danube, 2.36 in. per mile after the cut-offs were made, is so unusual that few comparisons are possible. Even those of the lower Mississippi, above its tidal portion, and the delta part of the Po are about twice as great. Apparently the only stream which is really comparable is our own Illinois River between Utica and its mouth at the Mississippi; in this gently winding stretch of 230 miles the average low water slope is 1.68 in. per mile.

The extraordinary characteristics of the Tisza, as outlined, indicate the feasibility of its rectification as a part of the general plan of flood protection of its great valley; although this procedure ordinarily produces such serious effects upon the regimen of a stream that its extensive employment would be hazardous. Even in the case of that river the increased discharge capacity as developed actually resulted in relieving its upper portion to the detriment of the lower part which the flood waters reached more rapidly than before, producing a partial engorgement when approaching the region where the natural conditions had not been equally modified to accelerate the flow. Among the many catastrophes due largely to this cause is that of 1879, when the important city of Szeged was overwhelmed by the most severe flood that had ever menaced it up to that date. Above the profile of Fig. 87 are hydrographs showing the relative durations of a high water stage in the upper and lower parts of the river, indicating a period for the latter more than twice that of the upper portion at similar stages. At Nameny it exceeded the elevation of its natural banks, 28 ft., for only three days; while it remained above the level of the natural surface of the plains at Szeged,

whose general elevation is about 15 ft. above low water, for a period of seventy-six days. It is the difficulties and dangers attending a long period of saturation of levees which, next to their being overtopped, causes the most serious danger to their integrity. The employment of cut-offs to shorten the Tisza was found to produce an increasing tendency of the accelerated currents to erode its banks in what is described as the endeavor of the river to resume a widely curving course. This has necessitated a continually expanding system of bank protection to hold the stream in its designed location.

Cross-sections of the low water and flood flows are also shown in the upper part of Fig. 87 (p.311), for three widely separated points of the river. The low water discharge at Szeged is 7000 cu. ft., and that of high water 134,000 cu. ft. per second. The range of stage is about 26 ft. at Tokaj, 29 ft. at Szeged and 20 ft. at Titel. The height of the river banks seems to vary from about 25 ft. in the upper part to about half this in the lower portion, so that the depth of water against the levees at times of flood varies from a small amount to a general maximum of 12 or 14 ft. The standard distance apart of the levees is 2500 ft. above Csongrad and 2625 ft. below that city; but local conditions frequently caused considerable variations from these values. The total length of embankments is given as 2062 miles.

The dikes of Holland have long been famous as admirable examples of the effectiveness of embankments in preventing the inundation of the lowlands which they defend. The condition in that country is remarkable because such a large proportion of the protected territory is below even the low water surface of the rivers, and also because of the persistent exclusion of the ocean from extensive areas which lie below the surface of the sea. The region which is safeguarded from floods by the river dikes is represented by Nolthenius as having an area of about 2100 square miles, and that which would be lost except for the great sea dikes is given as about 4800 square miles. While a considerable district is common to both estimates it is evident that the total area that is protected by levees from inundation is considerably in excess of 5000 square miles, some of which is 15 ft. or more below sea level.

The lowlands of Holland have, like those of the Po valley, furnished occasion for the employment of defensive embankments for an unknown number of centuries. The present system has been a matter of irregular growth through several hundred years; but

general success in giving an effective flood protection can hardly be considered as satisfactorily attained until after their systematic development was made a national policy in the middle of the last century. The service required of them is especially severe because of the saturating influence of the waters upon the banks and levees, resulting from the unusual lowness of much of the protected territory; because of the extent of these defenses along the sea or the broad estuaries which are subject to the relentless attack of the waves; for the reason that the danger of the formation of ice gorges is a very serious menace in that northern climate; and because so much of the material available for their construction, as well as serving for their foundation, is a poor quality of earth which is sometimes so defective as to be properly termed treacherous. And yet, although numerous breaches have occurred and many of the inundations have been calamitous in their proportions, reconstruction and improvement of the system through the centuries has been carried on with such determination and persistence that the breaks have been continually reducing in number until they are now practically eliminated; and the dikes of Holland constitute the most impressive example of the successful use of levees for flood protection.

The low water slope of the Meuse is about 25 in. per mile for the first 50 miles within the frontier of the country, about half this for the next 20 miles, and about 6 in. per mile for the lower 70 miles; that of the 40 miles of the Waal above the tidal influence is also approximately 6 in. per mile; and the same rate of fall exists in the upper 30 miles of the Yssel, but reduces to about 4 in. on the remaining distance of about 40 miles. The low water discharge of the Meuse is about 700, its flow at mean stage is about 4000, and at time of flood, 85,000 cu. ft. per second; corresponding figures for the Yssel are 1700, 8000 and 57,000; and for the Waal, 21,000 cu. ft. per second at low water, 53,000 at mean flow, and 220,000 at flood stage. The extreme range between high and low water is about 25 ft. in the three principal rivers of Holland. However, during the ages through which these streams overflowed the lowlands, the deposit of silt from the sedimentary waters has gradually elevated the banks of the streams; and throughout the recent centuries, during which the bordering dikes have been constructed, a similar raising has continued on their river side. The result has been that the elevation of the banks above the minimum stage generally lies between 10 and 15 ft.; so that the height of water against the river levees rarely exceeds 15 ft. The banks are thoroughly protected

against erosion, so that the position of the embankments is permanent.

The distance apart of the principal dikes, which approximately parallel the course of the rivers, varies from about a half mile to three times this distance or more. There are also many lower levees between the main dikes and the banks of the stream, whose purpose is to protect the marginal meadows from overflow from all except the high floods, that are similar in general characteristics to those of the River Po. In addition to these two classes of embankments there is a third system of large extent and great importance, situated within the various regions protected by the main dikes from inundation. The principal reasons for the development of this interior system are found in the local requirements which are largely peculiar to that country. Some of them have been constructed to restrict the areas flooded by the system of overflows existing at occasional points along the principal dikes, which were planned to relieve the main rivers at times of excessive volume of flow as mentioned in Article 18, and so reduce somewhat the flood height of the streams. Others have been built to localize the inundated area when a breach might occur in the main system, and so minimize the possible damage to that densely populated and very valuable territory. Many form the artificial banks of the network of auxiliary waterways of the country, most of which are true canals, but some are old river courses which have long since ceased to be freely flowing streams, being separated from their original uninterrupted connection with the rivers by dams and locks and so converted into stretches of still water. And finally, a great number of these subordinate interior levees serve as the artificial embankments required in connection with the extraordinary drainage system developed to supplement the advantages of flood protection secured by the main dikes. Interior embankments often serve two or more of the distinct purposes just mentioned, those of slack water navigation and drainage occurring together with especial frequency.

The flood protection of the lower Mississippi valley is chiefly notable because of the magnitude of the elements involved in its accomplishment. Bordering the length of river below the confluence of the Ohio, exceeding 1000 miles, there is an area of nearly 30,000 square miles that was naturally subject to overflow by the high waters. The stream itself is great. Its extreme low water discharge is about 65,000 cu. ft. per second; its average bank-full volume, from 1,000,000 to 1,200,000; and its flood flow at

times exceeds 2,000,000 cu. ft. per second. Its surface slopes are also large throughout the greater part of its length, when considered in connection with its volume, averaging 4.6 in. per mile for the 600 miles above Vicksburg. In the 160 miles from the latter city to the mouth of the Red River the total fall at low water is hardly 37 ft., while in the remaining distance of about 300 miles to the Gulf it is only 3 ft.; but at high water the slopes below Vicksburg are increased to an average of 3.2 in. to the Red River and 2.3 in. per mile below that point. Consequently the mean velocities at flood stages are also high and severely tax the resistance of works of control, and actual velocities as great as 15 ft. per second have been measured.

The high water surface has attained an elevation of from 45 to 55 ft. in the upper 660 miles of this part of the Mississippi River, and at places it has exceeded the larger figure; from the mouth of the Red River it gradually reduces from 53 ft. at that point to 0 at the Gulf. The general height of the alluvial river banks in the upper three-fourths of this valley is from 30 to 40 ft. above the low water surface, but becomes less as the Gulf is approached. Consequently the elevation of flood waters against the levees is from 15 to 20 ft. above their base in many places, and in others it exceeds the latter figure. As in all other systems, there are occasional points, such as where ravines reach the river, at which the height of the artificial embankment is perhaps doubled for short distances.

Because of the slow and irregular development of the Mississippi levee system from the small beginnings at New Orleans nearly two centuries ago, in detached portions that have been gradually extended and strengthened, which have been usually located with regard to the local purpose of flood protection alone and so with inadequate reference to the course of the river adjacent, there is a great variability in the high water widths of the stream between levees. The fact that nearly all of the river banks are still unprotected from erosion leads naturally to their general location several hundred feet from the edge of the river so that they will be safe from destruction for many years. Yet the typical condition of alluvial river valleys illustrated in Fig. 88,[1] showing the characteristic downward slope of the ground away from the immediate banks produced by the slackened velocities of the overflow causing the deposit of the greater part of its silt in position near to the margin of the stream, but which in other parts of this territory is often less rapid than is indicated in

[1] Report, Chief of Engineers, U. S. A., 1895, p. 3956.

the illustration, leads to the building of the embankments as near to the river as is feasible in order to minimize their height. They are very frequently from 1 to 2 miles apart, but are sometimes so located that the flood surface is 5 or 10 miles wide; as where the bluffs reach within a few miles of one river bank making it unnecessary at present to build a levee at all on that side, or where one or more loops of the river have made it much more economical to locate the embankment from one bend across the intervening neck of land to the adjacent curve than it would be to follow the windings of the river and return to a point comparatively near the preceding loop on the same side.

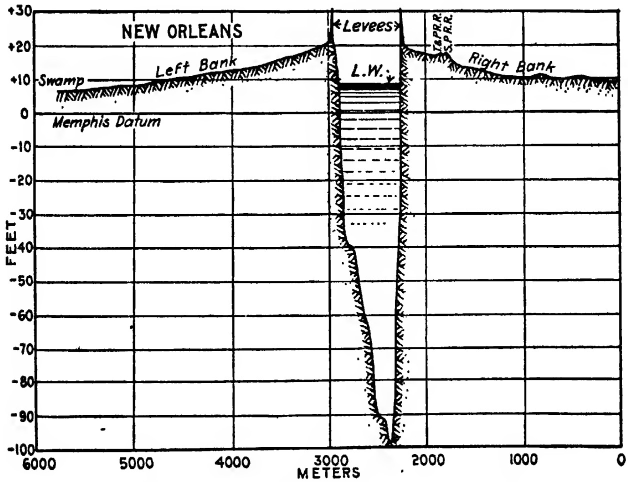


FIG. 88.—Cross-section of the Mississippi valley at New Orleans.

In the 1500 miles of length of levees now bordering the Mississippi River there was, in 1913, a total volume of 251,210,000 cu. yd. of earth. The expenditure by the federal government has aggregated $28,453,391 on about 125,000,000 cu. yd. which it has placed in them from the beginning of its participation in their construction in 1882, until 1913. The amount devoted by state and local authorities to their share in this main system is not published; but estimating on the yardage basis and making allowance for the great aggregate mileage of lost or abandoned levees, their expenditure during the same length of time would be in the vicinity of 30

percent greater than that of the federal government. This procedure omits consideration of the cost of embankments constructed in the century and a half preceding 1882, the amount of which is entirely unknown; but as the greater part of the work was quite local in character, and especially because most of that early construction has long ago disappeared in the river, its omission is unimportant. The total expenditure on the levees of the lower Mississippi river, during the more recent period of systematic development, may therefore be considered as about $65,000,000. Nor is this system yet complete. It requires extension in places where no levees now exist and enlargement throughout the greater part of its extent where they are deficient in height and cross-section. Even if an equal expenditure should be still needed to make them adequate for their service, the average cost per acre for protection from inundation would be only a fraction of that which has been incurred in similar enterprises upon many foreign systems. For example, Holland is proceeding with the projected reclamation of an additional area of 817 square miles from the Zuider Zee, at an estimated cost of $75,000,000. Surely the present economic importance of the vast territory of the lower Mississippi valley, which requires flood protection, amply justifies the expense necessary to make it effective; and the future development of this region will then warrant an evolution of the levee system in supplementary detail somewhat similar to that already existing in other parts of the world, but which now is subordinated to the need of perfecting the great primary defense against general inundations.

79. The Effect of Embankments upon the Regimen of Rivers.— Levees may be regarded as artificial structures supplementing the natural banks of alluvial rivers, to at once elevate them above the high water surface of the stream instead of awaiting the accomplishment of this result by sedimentation from periodic overflows through the lapse of countless ages. Such a confinement of a river to a flood channel of restricted width necessarily produces an increase in the elevation of the high water surface. Observations that the building of levees has raised the flood plane from 3 to 8 ft. at various places on the Tisza, 6 to 9 ft. on the Loire, 3 to 7 ft. on the Po, 4 to 8 ft. on many parts of the Mississippi during the past thirty years of active construction, and many other published facts and tables of similar import, necessarily involve a considerable lack of precision in their indications because of other causes contributing to these variations, such as differences in the volumes of flow which are com-

pared; but the general and systematic increase of flood height is only the visible indication of an effect that is inevitable. The practical significance of this universal result lies in the consequent need to fix the crest of levees safely above the additional elevation which the flood waters will attain.

The marked increase of flood heights which characterizes the restriction of a stream between artificial embankments has unfortunately led many to believe that this effect is necessarily accompanied by a rising of the river bed. The plausibility of this theory seems particularly indicated by the very general experience that levees, that had been sufficient for a time, have subsequently had to be raised to make them adequate; and later conditions have often necessitated the repetition of such enlargement. But to explain this situation by assuming that its cause is necessarily a rise of the river bed ignores other agencies which also produce an elevation of the flood plane. Among the several conditions of this kind are two which are always present. The first exists throughout the period, extending through many scores of years on some of the great systems, during which the progressive restriction of the river by levees has been in course of accomplishment by reason of their construction, enlargement and extension. Not only does this effect follow the more definite confinement of the stream at the locality in question, as when an embankment is built where none previously existed or when one is enlarged to confine the highest floods instead of merely serving the former purpose of excluding only the more moderate ones; but it also occurs in response to changed conditions above or below the place whose high water mark is under consideration, producing a modification in the run-off phenomena. An example of the former was the situation at Memphis, where the range of stage increased about 5 ft. during the ten years of most active construction of the levees of the St. Francis River basin on the west side of the Mississippi; and a case of the latter sort is the rise of the high water level at Carrollton of about 4 ft. in the latter half of the last century, largely due to the fairly effective progress in restricting the floods to the river itself in the distance of 957 miles above, which thus destroyed the former relief occurring to the lower part of the river when a considerable part of the high water volume remained in the great overflowed areas until the flood crest had passed onward to the Gulf. It is also true that as the levees are developed, and consequently the occurrence of crevasses becomes more rare, the effect is similarly to raise the high water level. This is evident from

the reverse phenomenon attending failures of sections of levees; as illustrated by such a typical experience as the break about 3 miles below Greenville in 1903, which immediately reduced the gauge reading 2 ft. at that city and which influenced the crest heights by lessening amounts for 45 miles above and 135 miles below. The second condition, which will always greatly affect the height reached by the flood plane at any place, is the extreme variability in the rainfall and run-off conditions in the watersheds of the different affluents producing aggregate effects that are never the same, as discussed in Chapter II. Therefore any direct comparison of flood crests is likely to be misleading unless it is based upon proportional volumes of flow, a procedure which has been more often disregarded than employed in published statements of opinion upon this feature of the question.

These and other considerations make it evident that there are many causes which may contribute to an increase of flood height in rivers. It is also clear that a more definite and independent proof is necessary as to whether or not the construction of levees does actually produce a rise of the river bed, especially as such a result would, in general, seem so contrary to the effect that theoretically should follow the concentration of flow into a narrower and deeper channel. The determination of this question is also most important because, if a rise of the river bed does occur, the embankments must be continually and indefinitely enlarged to keep their crests above the river which would also be constantly increasing in elevation. To obtain empirical evidence is by no means a simple proposition because the channel of an alluvial stream is so comparatively unstable and variable. It is deepening at one place and shoaling at another, widening in one locality and narrowing elsewhere, even at the same stage. And these contrasts in its channel-forming activities are so enormously accentuated by the variations in stage, producing corresponding changes in velocities, that local modifications may appear to either prove or disprove the truth of the assumption, depending upon the tendency at the place and time considered. A relatively small difference in volume of discharge, occurring at the two different dates selected for comparison, may mask or even reverse the indications which are relied upon to reveal the true situation. Yet it is perfectly possible to obtain this information by adopting methods of investigation which will minimize these confusing elements, and to determine any general effect of the kind, especially if it is of material amount. Of the numerous discussions that have

appeared, those will be given which bear evidence of definite reference to observed facts and of the most complete consideration of actual conditions.

In this country there has been a very extensive and careful comparison of a multitude of instrumentally located elevations defining the bed of the Mississippi River from Cairo to Carrollton, as determined in the original complete survey of 1880–83, with a similar series of more than three hundred thousand elevations in the same length of river, referred to the same datum plane as before and taken by special surveys made in 1894 and later years. The general conclusions concerning the effects of active levee construction upon the channel of the river are thus summarized.[1]

"The comparisons as a whole show a decided depression of the bed for a large proportion of its length and point to a general enlargement of the cross-sectional area of the stream below the high water banks. The results of the comparisons of so large a number of elevations justify the conclusion heretofore stated that there has been no measurable progressive elevation of the bed of the river during the period covered by the investigations cited."

Another independent general method of judging this question is offered by a comparison of the relative heights of the low water surface in 1894, at a score of gauges distributed throughout the Mississippi River below Cairo, with the lowest level which any previous low water surface had attained at these same points.[2] Ninety percent of these places showed a lower elevation of low water in 1894, although the affluents except the St. Francis River showed higher stages than at some previous date. The average depression, as thus indicated, amounted to somewhat more than a foot; but a considerable part may be but a temporary manifestation. Similar comparisons of the partially leveed upper portion of this part of the river with the extensively embanked lower part showed that the depression of the low water surface of the latter averaged about 19 in. greater than that of the former in a mean period of seven years. While indications such as this are not definite yet they fortify the conclusion, already quoted, that the general effect of levees upon the Mississippi is a tendency to lower its bed, rather than the opposite influence.

Because of the use of levees on many of the rivers of France, such as the Loire, Rhone, Garonne and Seine, the engineers have been

[1] Report, Chief of Engineers, U. S. A., 1912, p. 3717.
[2] Report, Chief of Engineers, U. S. A., 1895, p. 3624.

constantly on their guard to detect any systematic indication of a rise of the river bed; but none has been discovered. The Elbe, Oder and Rhine of the German Empire have also long been leveed, and similar studies have been made upon them. The result is evidence of change so slight as to be almost negligible in those cases where any general tendency is observable. For example, this question has been the subject of study by Fijnje, Hagen and other authorities in considering the indications furnished by records of the latter river. Comparative low water records at Cologne for about a century suggest a probable rise of the bed of perhaps $\frac{2}{3}$ ft. in a century; while at Düsseldorf there seems to have been a lowering at about the same rate, and a somewhat greater depression of the river bed at Emmerich. The average lowering in the Tisza River from 1840 to 1891 was 4.3 in.; but as this effect was a combined result of the construction of levees and the shortening of its course, it is not significant. The most persistent citations advanced by those who advocate the theory of a progressive rise of the river bed as a result of the employment of levees to confine the high water flow are those of the Po and of the Yellow River of China. The latter case may be dismissed from consideration because published statements of its behavior in this regard, on both sides of the controversy, are more in the nature of impressions; no thorough, scientific investigation of the question having ever been made on the Yellow River.

The Po appears to offer an unusual opportunity for the determination of the point at issue, by reason of the length of time covered by the development of its levee system which has been available for systematic examination of effects of this nature. Notwithstanding this situation, there have been many superficial statements which have received considerable publicity; and some published reports of authorities have been used as the basis of unwarranted deductions; as that of Prony, whose statement that at times of flood "the surface of the river was above the roofs of the houses at Ferrara" has been perverted by some into evidence of a considerable progressive rise of the river bed, due to the existence of the levees; and reports of such men as Lombardini, Baumgarten, Zendrini, Manfredi and Cuvier on certain details have been distorted to imply a general condition which was not at all warranted by the observed facts. Consequently there exists a great lack of agreement in published accounts, with a corresponding confusion with regard to the actual situation disclosed by the various investigations that have been made. The uncertainty is not surprising, however, when one realizes the

slow rate at which such a tendency develops, and especially when it is found that nearly all the observations are based upon the increase of flood heights, which is the combined result of several different influences. Even on this basis, but counting upon the exclusion of other contributing causes by comparing the heights of only those moderate floods which were held within the low levees completed before the period investigated, Lombardini's extensive researches indicate a general rise of the river bed averaging less than 5 ft. in seven centuries.

The most satisfactory class of observations, made upon the Po, was that in which the relative elevations of the low water surfaces at different times were compared. Of these the one at Quatrelle is typical both of the method and of the character of the prevalent effect disclosed. By reference to the old sill of the sluiceway at that place there is shown a probable rise of bed of 8 in. in 208 years. Other results vary in magnitude, but they are all small. Probably the most discriminating analysis of this question is that of Comoy,[1] in which he shows the absence of indications of a rise of bed in the Po except within about 90 miles of the sea. Such evidences are therefore confined to the lower portion of the stream, which is subject to a condition that is not at all typical of the greater part of the length of the Po, nor of that of other embanked streams. This special circumstance is the progressive extension of the mouths of this river into the tideless Adriatic Sea, which appears to have exceeded 30 miles during the Christian era, has amounted to about 9 miles in the last two centuries, and continues at a rate exceeding 200 ft. per year. The advance of the shore line at the river's mouth, and the resulting gradual reduction in surface slope and the raising of the water surface throughout the extent of this influence, is considered entirely sufficient to account for the slight rise of bed that characterizes the lower portion of the Po.

The evidence therefore appears to not only dispel the illusion that the construction of levees is typically followed by a progressive elevation of a river's bed, but to verify the clear import of hydraulic principles that the tendency is quite the opposite. This general conclusion must not, however, be understood to be universally true. Local exceptions occur as a result of special conditions; such as exist where a stream, heavily loaded with sediment, reaches a part of its course where the slope becomes so much less that its transporting

[1] Paper 266, Tome XX, Third Series, Annales des Ponts et Chaussées, pp. 257–304.

power is no longer capable of carrying the load, and the embankment of the river so greatly restricts the area available for the inevitable deposit that its more rapid accumulation in the river bed accentuates the phenomenon. Such conditions exist in the lower portion of the Reno and Adige of Italy, as well as at parts of other rivers of the world. In fact, there are also various circumstances under which the process of adjustment of a river to a modified regimen involves a local rise of its bed. Furthermore, the alternating fill and scour, which accompanies the rise and fall in stage at any point of an alluvial stream is of course a manifestation of responsive fluctuations that have nothing to do with permanent tendencies. The general effect of levees in causing a lowering of the bed of the stream is, however, a result which is usually so very gradual in its evolution that it actually must be considered a factor of minor importance in connection with the construction of levees.

A relatively more marked effect of embankments, which is more rapid in its progress although really quite moderate in its rate of development, is the tendency to produce a greater uniformity of the channel dimensions. This is most clearly indicated in the comparative surveys of the Mississippi river already mentioned. In general it was found that the bed of the river at the shoal portions was lowered, and in the pools it was raised. There was also evident a systematic enlargement of the river channel, in its gradually developing endeavor to respond to the increased duty put upon it, which occurred principally in that part extending from the low water surface to the top of the banks. Typical values of the average amount of enlargement are 3.2 percent between the mouth of the Arkansas River and Vicksburg in fourteen years; 2.2 percent between Scot Bluffs and Donaldsonville in fifteen years, but 3.7 percent in a two-year period which includes the double influence of the engorging effects produced by two seasons of unusually moderate stages preceding the first of the comparative surveys and by the flushing effect of the extraordinary flood of 1897 which preceded the later survey; and 1.1 percent between Donaldsonville and Carrollton in four years, a result which is similarly affected by the scouring flood of 1897. Local changes of much greater magnitude have been observed; as at Wilson's Point, where the increase of area amounted to more than 16 percent in nine years.[1] Another kind of evidence,

[1] Transactions, American Society of Civil Engineers, Vol. 35, p. 369; correspondence on "The Discharge of the Mississippi River" by William Starling.

which is quite marked, is the frequently observed fact of a shoaling and contraction of channel below crevasses, with a subsequent restoration of original conditions after the breaks had been closed. These typical examples illustrate the fact that progressive but slowly developing tendencies of a general nature are accompanied by fluctuating values of such irregular occurrence that a very discriminating investigation is often necessary in order to disclose the characteristic details of the general effect sought.

The prevailing trend of levees toward producing an equalization of channel dimensions and an enlargement of the cross-section is not only so small that it is easily obscured by other influences, such as the more perfect confinement of the floods or the fluctuations in volume of discharge; but its rate of development is ordinarily so slow that the consequent lowering of flood heights is similarly masked and is generally disappointing to those who have expected these results to be quickly apparent and strikingly favorable. Yet there are occasional instances in which the rapid progress of such effects has been observed, as on the Atchafalaya and Red Rivers. With regard to the latter it is stated[1] that

> "It is an undoubted and incontrovertible fact that on that part of Red River in Louisiana where, for some fifteen years past, levees have been maintained, and the waters of the river have been regularly confined to the main stream, the width of the stream has almost uniformly more than doubled itself, and the increase in depth has been generally not less than 2.5 ft., and in places much more. When first confined, the plane of high water was in many places materially raised, but it is now decidedly lowering, there being apparent in the last two or three high waters, a lowering of flood levels, of as much as 2.0 ft. along those parts of the river confined by levees."

80. Their Influence upon Navigability.—The particular feature of the effect of levees on the regimen of rivers, which intimately concerns them as transportation routes, is the character of their influence upon the navigable channel depths. It is evident, from the preceding paragraphs, that the regularizing tendency directly affects this consideration in the gradual raising of the bottom of the p and the lowering of the crests of the shoal places. The former de opment is ordinarily of no importance in this regard; but the la attacks the precise places which require deepening. Unfortuna

[1] Transactions, American Society of Civil Engineers, Vol. 58 (1907) p. 23; "The Atchafalaya River; Some of its Peculiar Physical Characteristics," by J. A. Ockerson.

for the effect desired, this natural modification usually results in so considerable a lowering of the water surface that the greater part of the increase of navigable depth, which otherwise would be secured, is lost.

The expected efficacy of levees in contributing to the improvement of the channel may be considered as, in principle, corroborated by experience; but the average rate of this irregular development is, in general, so extremely slow that its serviceability in increasing the depth of water on the normal bars has ordinarily been very small in its quantitative effect. However it is believed that as the levees of a river are perfected, especially with regard to the desirability of locating them in approximately parallel lines which shall follow the general windings of the stream, their beneficial influence upon the navigable channel becomes more marked. This expectation is encouraged by experiences in Holland and in some other countries of Europe, where especial attention has been given to the advantageous location of embankments with respect to their effect upon the channel of the river as distinct from their function of protecting from overflow. For example, the standardization of widths of the Yssel, that of the high waters being secured by dikes which substantially parallel the sinuosities of the river, is credited with producing the result of an unaltered bed during floods. Such an effect in an alluvial stream averts a substantial part of the impediments to navigation because of the resulting elimination of shoaling bars which typically are built up at times of high water with the result that the subsequent restoration of favorable low water conditions, as the river falls, is more or less incomplete.

All things considered, it seems probable that the greatest service of levees to navigation is preventive, rather than corrective, in character. The latter quality sometimes produces a marked deepening, often none, and in the general average a slight progressive increase of the limiting channel depths, as already indicated; but there are occasions when they prevent the serious local deterioration of the channel. This occurs particularly at flood stages when, if it were not for the embankments, a considerable part of the discharge would flow across the lowlands instead of following the course of the stream. The inevitably resulting distrubances in the regularity of flow and the retardation of the channel current so reduce its transporting power that more or less extensive deposition of sediment from the surcharged waters occurs, which becomes a serious matter when the place in question is a shoal part of the river. Such

an effect is avoided where levees are provided to keep the entire volume constantly flowing along the course of the channel; and the great advantage, so gained, is often not realized because the actual service rendered by their presence is ordinarily not in evidence while they stand. Yet it did appear in connection with the system of overflow relief of Holland that is gradually being abandoned; as, for example, the diversion of a part of the flood volume of the Waal at places of withdrawal

"Causes an abrupt change of direction of the current and a loss of volume, hence diminishes the velocity of the main stream, brings about deposition of silt, a rise of the bottom and a shallowing of the water, and therefore promotes the formation of ice gorges in the Waal, the evil of all most dreaded in time of flood, as endangering the dikes above."[1]

The quantitative amount of this effect of course varies greatly with local conditions, and the measurement of it necessarily includes other factors of regimen which are simultaneously acting. Yet there are cases in which the result in question is apparent, as in that of the Nita crevasse of March, 1890.[2] A survey was made in the following September, and with it was compared the result of a second survey made in June, 1891, the broken section of the embankment having been rebuilt during the low water season of 1890. Although this comparison does not include modifications taking place during the first six months after the crevasse occurred, the amount of enlargement of the river channel during the succeeding nine months elapsing between the two surveys, averaged about 6 percent for the seven sections distributed through a distance of 2½ miles above, and about 14 percent for the eight sections extending from the break to an equal distance below. As local mutations of channel conditions were also apparent, a conservative estimate of the original deterioration of the channel caused by the crevasse might be approximately represented by the difference between the values just given, which amounts to about 8 percent. A notable example of the same tendency is that of Cubitt's Gap, near the mouth of the Mississippi River. This produced a progressive shoaling, in the 3 miles below the break in the left bank, averaging almost 4 ft. in a period of about ten years.[3]

[1] Transactions of American Society of Civil Engineers, Vol. 26, p. 643, "Some Notes on the Holland Dikes" by William Starling.

[2] Report, Chief of Engineers, U. S. A., 1894, p. 3066.

[3] Appendix No. 10 of Report of U. S. Coast and Geodetic Survey, 1880.

81. The Development of Great Levee Systems.—The extensive construction of embankments to protect lowlands from the floods of rivers is always characterized by a comparatively slow progress toward the attainment of the perfected system. This is partly due to the impossibility of exactly anticipating the height which the flood waters will attain when confined, but is mainly the result of the deliberateness involved in the accomplishment of all great undertakings and of the economic need of avoiding extravagance in their dimensions and consequently in the expenditure. The dikes of Holland and the levees of the Po and other rivers of northern Italy have been in course of construction for hundreds of years; and their systematic enlargement, repair, and reinforcement, prosecuted with especial vigor for the scores of years during which the control of the main system has been a governmental charge, have been accompanied by a diminishing number of failures and an increasing reliability of service until their adequacy is comparable to that of other public undertakings in securing the intended results.

A similar evolution has characterized the system in the valley of the Tisza River. As in the majority of similar enterprises, the works were first begun in a disconnected, inadequate way by interests that were not properly coördinated. It was soon apparent that a systematic organization and prosecution of the work was essential to a successful outcome, and its entire control was consequently assumed by the national authorities. Under the general direction of a government commission, and later under the ministry of agriculture, the rectification of the river was effected at the public expense, as well as the supervision of the levee construction by twenty-nine local "Companies" that were organized for the latter purpose.[1] These companies were reimbursed by taxes assessed by law against the protected lands and other interests concerned, the apportionment being made on the basis of the benefits received by each. In a few extraordinary cases they were given subsidies from the government. It thus appears that the general rule followed in these Hungarian works conformed to the principle of making the expense of the improvements of the river channel a charge upon the national exchequer; while that of levee construction, an enterprise primarily for the purpose of flood protection of the valley, was met by the local interests directly benefited in this way, by an annual charge ordinarily distributed through a period of fifty years. The increas-

[1] Special Report of the Ministerial Counsellor, Herrich Karolyi, 1873.

ing efficiency of the embankments is also striking. Previous to 1867 the progress of construction was slow and unsatisfactory; but after that date the work was actively prosecuted. The marked progress in the effectiveness of the levees is indicated by the fact that, in the last century there were 692 crevasses, flooding a total area of three million hectares; while "since the embankments were increased to their normal dimensions, only two such breaches have taken place, the extent of the thereby inundated ground being 12,000 hectares."[1]

Previous to the middle of the nineteenth century, the levees of the Mississippi valley had been in process of slow development from the first small embankment, only about 4 ft. high, built at New Orleans soon after its settlement, to those of larger proportions of later years. They were also gradually extended northward by the efforts of communities, landed proprietors and other local associations, until they had reached the mouth of the Red River, with considerable lengths of disconnected levees existing as far north as the Arkansas River. Yet so elementary were they that only the most moderate floods were excluded; as may be judged from the fact that a survey in 1851 showed that their average height, between Red River Landing and New Orleans, was less than 5 ft. About this time a considerable impetus was given to levee construction by the grant of the overflowed lands to the states in which they were situated, but this improvement was only temporary and actually was followed by a serious retrogression during the military operations of 1861–5. Again the development of the system was undertaken by the resolute county and state levee boards until, in 1882, there was a fairly continuous line from Memphis down-stream on the east side and below the Arkansas River on the west bank. Yet so inadequate had been the available resources that the average height of the levees then existing was less than 8 ft. The unusual flood of 1882 overwhelmed the system and devastated the valley, causing 284 crevasses aggregating 59 miles in width, and inundating more than two-thirds of the supposedly protected lowlands. In that year the federal authorities began the allotment of funds to aid in the construction of embankments; and since that time their enlargement and extension have been prosecuted with increasing vigor by reason of such combined support to which the states, levee districts and other local organizations have contributed a total that is more than 30 percent in excess of that

[1] Page 8 of Brochure 2. Section 1, Question 5, of the Eleventh International Navigation Congress.

of the federal government, but which has been slightly less than the national allotment for the last four years.

For a dozen years after the flood of 1882 the efforts were mainly directed toward the repair and strengthening of the levees. Their extension into new territory was also gradually taken up, the line on the right bank from Helena down being closed by 1897; and the last great stretch, that along the front of the St. Francis Basin, was so far finished that it completely excluded the rather moderate flood of 1906. Their progressive efficiency during the evolution of the system is indicated by the gradual reduction in the number of crevasses. In 1883 there were 224 having a total width of 34 miles, and in 1884, 204 with an aggregate width of nearly 11 miles. Similar figures for 1897 are 38, totaling somewhat more than 4 miles; and in 1903 there were 6 breaks flooding less than one-fourth of the valley. In that of 1912,

"After a lapse of nine years with but slight damage to the levees from floods comes the greatest of all recorded floods on the lower Mississippi River. Twelve crevasses occurred in the controlling levees between Cairo and New Orleans. Only two crevasses occurred in the controlling levee on the east side of the river above New Orleans. No crevasses occurred in the Upper Yazoo Levee, 98½ miles long, nor in the Ponchartrain levee, 126 miles long. One crevasse occurred in the Lower Yazoo Levee, which has a length of 188 miles. At Point Pleasant, which is at the head of the Lower St. Francis Levee district, the flood poured through a gap which has been left open for several years on account of the refusal of landowners to grant the necessary right of way," etc.[1]

After every great flood of the lower Mississippi valley there are many who, overlooking the increasingly effective protection afforded, declare the levee system a failure and advocate other measures of relief. Although reservoirs, channel rectification and enlargement, relief channels, or other methods are available for this purpose in many instances, embankments must constitute the essential defense of this lower valley for reasons discussed in detail in Chapter II. Even expert judgment has occasionally advocated, at recurring intervals, the construction of levees sufficient to restrain only the moderate floods.[2] This would mean the permitted inundation, at frequent intervals, of three or four times the area of lowlands submerged in the record floods of 1912 and 1913; and the value and importance of the vast interests of the valley would not sanction this

[1] Report, Chief of Engineers, U. S. A., 1912, p. 3721.

[2] *e.g.*, Journal of the Association of Engineering Societies, Vol. 3, pp. 169–181. (1884.)

alternative, nor would the experiences of similar undertakings abroad substantiate such a course, especially as the system is still far from complete.

The progress in the development of the levees is also shown by the yardage values, which amounted to less than thirty-three million in 1882, about ninety-nine million in 1895, one hundred ninety-seven million in 1905, and two hundred fifty-one million, two hundred ten thousand cu. yd. in 1913. The facts that one-third of the total levee mileage is now below the provisional grade, and that they are still deficient in section for a large part of their entire length, certainly give cause for rejoicing that these unfinished embankments have withstood the extraordinary tests of the recent unparalleled floods as well as they have done. The situation is similar in principle to any great undertaking, which involves a large expenditure that is generally supplied in a very deliberate fashion, and which should be adapted in detail to the conditions of service as they appear during the period of development. For example, it is the general experience in railway construction to find that many years and even decades are often necessary to transform a crude roadbed and inadequate track into so effective a condition that accidents due to such deficiencies are minimized. However, the rate of development of a levee system, like that of any kind of public works, can be largely expedited if the allotment of funds for its construction permits; and the margin of safety is in direct proportion to the massiveness of design, and therefore to the cost.

When it is noted that all the levees which failed in the unparalleled flood of 1912 were below the standard grade and section, with possibly one exception; that the main lines of defense were effective throughout the fifteen years from 1897 to 1912 except for the six breaks occurring during the flood of 1903, which reached a stage averaging higher than any previous one; and that the 3281 square miles of the Upper Yazoo levee district has had complete protection from overflow for the last seventeen years; it would seem that there is a very substantial basis for confidence in the adequacy of the protection of a completed system. The immediate necessity is the enlargement of the main levees to the provisional height and standard section proposed, with the strengthening of sections or of support where defective material has indicated this need.

82. The Drainage of the Protected Territory.—The method employed to manage the run-off from the lowlands of a valley, during

the continuance of the high water stage confined to the course of the river by the embankments, is chiefly dependent upon the value of such lands; while the details of such measures and the degree of efficiency attained by them are also affected by the topographic, climatic and other characteristics of the region and by such circumstances as the duration and height of the flood. The usual procedure is to pierce the continuous levee line, wherever it crosses a natural water course, by a culvert or a sluiceway, so arranged that the drainage may freely pass outward to the embanked river as long as its stage is below that of the branch; but so controlled that the direction of flow cannot reverse and flood the lowlands when the water surface of the main stream is the higher. In case there is an affluent of large size encountered, it is usual to build levees upon its banks also, up stream to higher ground; although occasionally these are omitted, allowing the flood to locally inundate the adjacent region, especially if the property values affected would not warrant the cost of protection, or when the general system has not reached the point in its development at which this local improvement may be advantageously undertaken.

Because the existence of any structure through a levee is a source of more or less weakness, the construction of the culverts and sluiceways is accompanied by precautions that assist in safeguarding the integrity of the embankments, such as so thorough a construction that an eroding leakage into the embankment is precluded, and the building of adequate masonry walls to protect the earth slopes at both ends. Their number is also generally reduced as much as is practicable. For the latter purpose ditches or canals are cut in a way to lead several water courses to a single outlet when feasible; as proposed, for example, for collecting the streams (that now cross the upper end of the Little River Drainage District) into a single diversion channel leading directly to the Mississippi River near Cape Girardeau, and thus excluding them from the district by a levee paralleling its lower bank. However, culverts for small branches and sluiceways for those of moderate size constitute the standard method of control. The former now generally consist of cast-iron or reinforced concrete pipe fitted with automatic valves opening outward, but closing when the water level outside is the higher. Sluiceways are of much larger capacity, and the flow through them is controlled either by vertically sliding gates or by those which swing horizontally; in the first case they are opened and closed by mechanical power, but the operation of the latter is largely

automatic. The number of culverts and sluiceways in Hungary averages about one for each 2 miles of levee line.

During the time that the outlets are closed to prevent the inflow of flood water the natural run-off, augmented by the seepage through the levees, collects at the lower levels of the protected area. When the flood period is protracted or the run-off of the tributaries is great, the accumulating volume of water awaiting discharge may itself become a serious menace. The remedy for such a situation is a pumping system which can be used in emergencies of this kind. Obviously such an installation is only justified in cases where the importance of the property to be protected is considerable.

The densely populated and very valuable lowlands of Europe naturally exhibit the most complete employment of extensive and thorough drainage facilities. Among them those of Holland and Hungary are conspicuous. In 1908 there were in the latter country sixty-five pumping stations in the valley of the Tisza and sixty-four in that of the Danube, whose combined capacity was 6100 h.p. Centrifugal pumps are used, the total number being 180, whose aggregate capacity exceeds 6000 cu. ft. per second. The cost of the pumping stations of the Danube was $1,000,000 and of those in the Tisza valley, about 50 percent greater.

The drainage system of Holland surpasses all others in the elaborateness of some of its characteristics. This is partly the result of the fact that so much of its territory is always below the level of the rivers; although that condition does not signify the need of constant pumping because at certain seasons the losses from evaporation and the growth of vegetation exceed the rainfall and seepage. The most striking feature is the frequent employment of vast storage basins into which the water is pumped from the lower levels, where much of it temporarily awaits discharge through sluice gates, culverts or locks at times when the water surface outside is not sufficiently low to permit its immediate release; but when the high stage is prolonged, the extra expense of pumping to an additional height is necessary. The storage basins are usually very long and narrow; being given this shape partly because it is found more economical and advantageous under the local conditions connected with the drainage problem, and partly because many of them are also used as navigable waterways. The area occupied by these storage basins varies from about 2 to 5 percent of the acreage served by them. Both steam and wind are used for power, there being hundreds of windmills in use; but the constantly available steam plants

have replaced a great part of the wind machines. Cornish engines and lift pumps are sometimes employed, and centrifugal pumps are greatly increasing in number; however, the old method of lifting water by means of inclined screws, by scoop wheels or by larger wheels carrying buckets upon the periphery, still prevails to a large extent. Such machines are often placed at two or three different levels where the total rise is considerable, because the advantageous lift of the wheel is limited to 5 or 6 ft.; while that of the screw may be more than twice as much. The capacity of many of the pumping plants of that country is necessarily very great, some of the principal ones reaching 1000 cu. ft. per second for a lift of 3 ft. An illustration of the economy secured by such temporary storage of the drainage waters is furnished by the instance in which less than 40 percent, of 17,000,000,000 cu. ft. yearly discharge from one reservoir, required pumping for its ultimate removal.

Many leveed districts of considerable size in the United States have very complete drainage facilities, including pumping installations; but both pumping and drainage channels through the embankments are practically unknown, as yet, in connection with the main levee lines of the lower Mississippi valley. The employment of culverts or sluices is avoided because they are believed to increase the danger of crevasses, and there is no occasion for pumping while the lower ends of the great basins remain open. This method is made possible by the topography of the valley, the general slope within each basin being away from the levees on the river bank; and thus the run-off from all the tributary area naturally seeks the channel of the stream which carries its surface drainage to the main river at those places at the lower end, already referred to, where the embankments are omitted. Of course the high waters of the Mississippi submerge rather extensive territory in the vicinity of such gaps; but the considerable seaward slope of the general surface of the valley limits the inundated areas to such an extent that their protection is of slight importance compared to that of completing the main lines of levees, and the entire exclusion of the Mississippi River floods from these subordinate regions must await the time when the local land values will justify the cost of the necessary complementary works.

An accessory service of considerable importance that is sometimes available to lowlands defended by embankments, is the convenient irrigation of agricultural products at times of drought when the excluded waters are above the level of the fields. The rice planta-

tions bordering the lower Mississippi are readily flooded by the use of siphons passing over the embankments of that river. The advantageous opportunity for supplementary irrigation was recognized in planning the levee system of the Tisza. The employment of temporary storage basins forming a part of the drainage system of Holland, as already described, offers peculiar facilities for the same purpose. Of course pumping ceases as soon as evaporation exceeds the rainfall; and, if the dry season is prolonged, water is furnished to the fields by reversing the direction of flow through the culverts and sluices, as needed. That this advantage is very substantial is indicated by the experience of the Rhineland district, in which the amount of water used for a considerable period averaged more than 9000 cu. ft. per acre each season, and in one particularly dry year it exceeded thirty thousand.

83. The Location and Dimensions of Embankments.—The urgent need of immediate protection from floods has often led to an unfortunate establishment of the levee lines; and the disadvantages of an incorrect location are perpetuated indefinitely as the embankments are strengthened and reconstructed. Their cost is so considerable that the avoidance of unnecessary expenditures is exceedingly important; and therefore a complete topographical survey is as essential as for an irrigation, drainage, or a railway project. Because the highest ground of an alluvial valley is usually found at the river, a minimum volume of earthwork will raise their crest to a certain elevation if they are built upon the immediate banks, a position which also protects a maximum acreage; but the former advantage is more or less counterbalanced by the fact that a wider spacing of the levees will somewhat reduce the flood height.

The relative durability of the embankments, as affected by the differing degrees of exposure of alternative locations to destructive agencies, is also a consideration of equal importance to that of first cost. Their construction along eroding banks inevitably results in their destruction in the length of time required for the river to reach their site, unless the recession is prevented by revetment or a change in the regimen of the stream at the place. The erosion of the levees themselves by the high velocities and swirling currents of the flood waters is another danger which may be actually invited by faulty position or alignment. Salient angles constitute an especially vulnerable point of attack, and are sometimes difficult to avoid at the place where a new section is joined to an old levee line. The more regular an embankment may be in its outline and the more grad-

ual its changes in direction, the less is its exposure to disintegration in this way. Another feature, sometimes of great importance, is the determination of the advantageous point of crossing the deep valleys of tributary streams. In all these considerations an intimate knowledge of the characteristics of the river when in flood, and a keen insight into the effects that will be produced by each detail of the project, are most valuable assets in securing an adequate location.

While giving chief place to the paramount advantage of defense against inundation, there are accessory interests that are important. A complete system of flood protection always requires adequate drainage arrangements; and, if these are extensive, their cost may often be favorably affected by the simultaneous consideration of this feature. The influence, also, that each detail of a proposed location will have upon the local and general regimen of the stream is receiving an increasing amount of attention particularly because the improved navigability of the river is indirectly involved to some extent. An adequate recognition of all the conditions and interests affected by a proposed location of levees will unquestionably insure their most complete serviceability, and therefore becomes the true measure of advantage of any project.

However, it is a fact that the direct requirement for flood protection generally dominates all other considerations. Topographical features and the exigencies connected with reconstruction of previously existing levees exercise a particularly persistent control upon the present lines, often rendering it economically impracticable to secure the approximate parallelism and regularity of outline that is theoretically desirable. This situation is well indicated by Fig. 89,[1] showing the main embankments of that part of the lower Mississippi, in the vicinity of the famous Greenville Bends, as they were in service six years ago. The spreading of the levees to include great loops of the stream as well as old lakes, and some portions of lines that have been abandoned because of the encroachment of the river upon them, are features which are especially noticeable. Piecemeal construction is responsible for a large part of the minor irregularities that are apparent. The changing position of the stream, due to the continuing erosion of its concave banks, is also indicated. The minimum width of foreshore on the Mississippi is intended to be a distance that will represent about twenty years of erosive

[1] Report, Chief of Engineers, U. S. A., 1908, p. 2776.

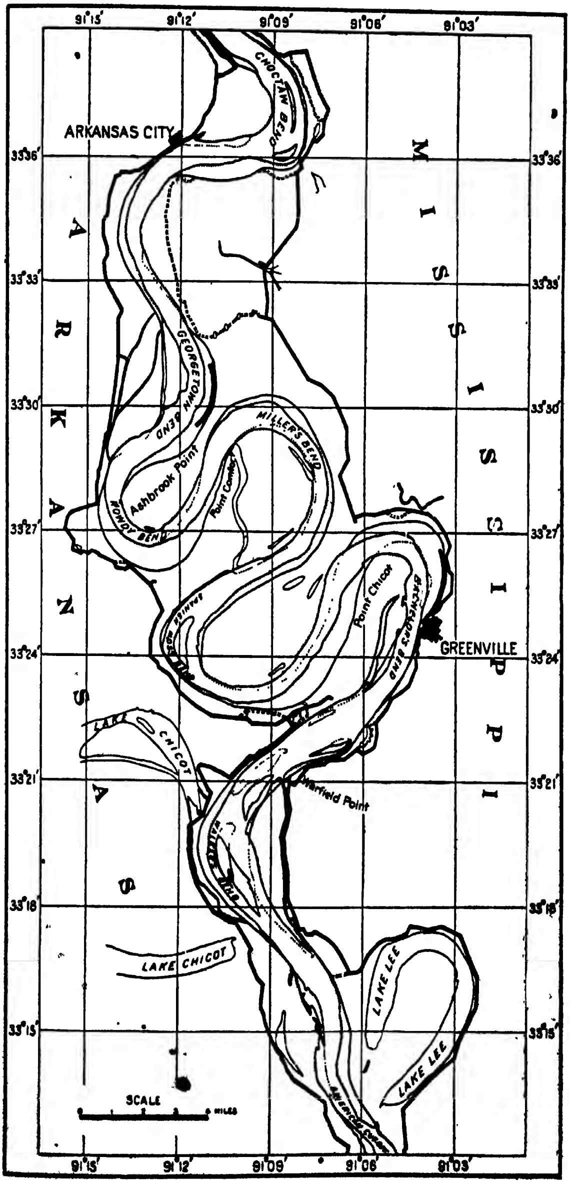


Fig. 89.—Levee lines near the Greenville Bends.

action before a caving bank reaches the levee. The width of batture on the River Terek, of the Russian Caucasus, varies from 150 to 1650 ft., depending on exposure; and its least width on the rivers of Holland varies correspondingly from about 250 ft. in small rivers to 600 ft. or more where conditions are especially unfavorable.

The dimensions of an embankment are primarily a function of its height, and this of course depends upon the elevation above its base that the confined flood waters will reach. When levees are first built the only available method of estimating the surplus rise, that will result from their construction, is that of hydraulic computation in which the volume of maximum discharge naturally flowing between the proposed levee lines is to be increased by the amount of flow heretofore passing over the protected area at the same time. The elevation thus obtained can be considered as only tentative, both because of the approximate character of such computations, as indicated by the general principles discussed in Articles 20, 27, 39, 40 and 41 of this book; and also for the reason that the additional volume to be provided for is very difficult to estimate, especially if the area from which it is to be excluded is quite broad or irregular in cross-section.

During the period covered by the rapid construction of levees there exists the significant opportunity to obtain with closer approximation the values of the observed volumes of flow and of the factors of the hydraulic formulæ, and so to attain a more correct determination of the extreme flood height. There also occurs the decided advantage of securing an independent estimate of that height, based upon the observed elevations attained by it under the new conditions, thus permitting the establishment of a provisional grade of much reliability. In fact, as the embankments become more effective in confining the flood flow, an increasing reliance is placed upon the indications afforded by the gauge readings as the stage reached at such times is adjusting itself to the new conditions.

However, the practical determination of a provisional flood elevation between levees, during their development, includes many considerations that require substantial evaluation. The velocities and volumes of the observed high waters are to be compared to those of the probable maximum discharge. The effect of crevasses in lowering the stage, where they occur, and in raising it unduly where the overflow returns, must be allowed for. This is generally done by extending the hydrograph of gauge heights, as a smooth curve

similar in form to that of analogous curves at somewhat lower stages when breaks did not occur, through the "date" abscissas representing the period of crevasse overflow or return, as substantiated by estimates based upon the volumes of flow escaping or returning. Modifications in height so produced often amount to 2 or 3 ft., and sometimes reach 5 ft. or more.

Wherever large affluents enter, the controlling conditions are so materially changed that the elevation of flood crests requires corresponding modification. Of course the maximum discharge below the junction point never reaches the sum of the maxima of each. This is due partly to the fact that the flood crest of a tributary rarely reaches the main stream at the same time as does its own crest, and partly to the really large storage effect of many rivers between levees. Both these facts are often imperfectly apprehended, the existence of a reservoir capacity at the place where it is most completely effective being frequently lost sight of; but that it is of very great significance is indicated by the volume concerned. For example, the storage capacity probably exceeds 30,000,000,000 cu. ft. for each additional foot of height at flood stages of the lower Mississippi, in the 400 miles of distance between the mouths of the Ohio and Arkansas Rivers. The combination of modified conditions caused by affluents necessitates, then, equivalent consideration in arriving at the elevation of flood crests in their vicinity and below. An example of a large effect of this kind is furnished by the fact that the oscillation in stage of the high water of 1912 at Memphis, where no large tributary enters the Mississippi River, was about 9 ft. less than at either Cairo (above) or at Helena (below) because great volumes enter from affluents near the latter cities; and there have been floods during which these differences were about 50 percent greater.

By choosing significant points along an embanked river for thus approximating to the probable elevation of the flood crest, the provisional high water grade throughout its course can be derived by connecting those elevations by a profile line whose general slope is modified by local influences such as widening or narrowing banks or levee lines, escape or return waters, and other conditions marking each locality and flood whose indications are all carefully considered in forming the collective result. The complexity[1] of such an investi-

[1] *e.g.*, See paper by William Starling, "The Discharge of the Mississippi River" in Transactions of the American Society of Civil Engineers, Vol. 34, pp. 347–492; and discussion of same in Vol. 35, pp. 305–370.

gation is very great because of the variability and uncertainty of the many elements involved, and yet much credence is due to conclusions derived from a thorough analysis of conditions, as indicated by the many instances of a very substantial agreement between the predicted flood heights and those afterward experienced.

After a prolonged period of observation while the levee lines are largely effective, during which the results of the varying flood volumes and other factors influencing the elevations reached by the high stages are becoming much more definitely apparent, the provisional grade is finally superseded by the standard grade whose reliability has been thus adequately established by extended experience. The latter is usually somewhat above the former, largely because the expense involved in levee construction seems to usually induce an inclination toward a conservative estimate. The added height of the standard grade usually ranges from nothing to 3 or 4 ft.

The crest of levees is always designed to be a few feet above the estimated surface of the maximum flood when confined between them, partly because of the possibility of greater volumes than any contemplated, and partly to compensate for such minor influences as local or special irregularities of high water, unexpected settlement or degradation of the embankments, etc. This excess height is about 2 ft. on the upper Rhine, Elbe, the least exposed portions of the Holland streams, and on rivers of Texas; about 3 ft. on the Po, Terek, lower Rhine, upper Tisza, Sacramento, and generally on the upper and lower Mississippi River; about 4 ft. on the larger rivers of Holland and along the front of the Yazoo basins of the lower Mississippi; 5 ft. on the lower Tisza, the particularly exposed river dikes of Holland, and the extremely important levees protecting the city of New Orleans, Louisiana; as much as 6 or 7 ft. in unusual circumstances, as on some of the embankments of the great interior basins of California which must successfully resist the wave wash of severe storms; and even 15 or 20 ft. in case of extraordinary exposure, as on some of the sea dikes of Holland. An unnecessary height is, of course, to be avoided; as the cost increases in proportion to the square of the height, for similar forms. But it is even more important to avoid a crest low enough to be overflowed, because then the resulting disintegration of the earth embankment is fatal.

The determination of the correct sectional dimensions of levees is based upon those fundamental principles that essentially govern the design of any earth embankment; yet the controlling conditions

are so generally different from those available for structures of limited extent, such as earth dams, that a quite distinctive type of cross-section is usually characteristic of levees.

The four principal considerations that affect this determination are the foundation conditions, the character of soil, the methods used in construction, and the adaptability to subsequent enlargement. As for the first, it is readily realized that levees must rest upon the natural soil of the river banks, whose supporting power is very variable, often doubtful in sufficiency, and sometimes treacherous. Subsidence has sometimes been so great that the volume of earth lost has formed a large part of the quantity placed. The only remedy usually available, in the case of inadequate bearing power, is to widen the embankment, as is the frequent practice; because a good foundation is generally too deep to reach. For example, the subsidence of the Lower Tensas levee, in the vicinity of Bougere, was terminated at its crossing of Boggy Bayou by reducing the side slopes of the lower four-tenths of height to only half the inclination above. Widening the base is also the usual recourse in minimizing the danger of leakage through a porous substratum, a source of weakness that sometimes ends disastrously; as in the case of the five breaks of the Lower Yazoo district in 1890.

The availability of material, the enormous volumes needed, and the necessity for strict economy compel the employment of that quality of earth which is available at the site. A cohesive, dense and practically impermeable soil is very important; but the alluvial deposits of river valleys rarely have these characteristics to the desired degree, and use must be made of what can be had. Clay constitutes the most adequate material and its quality is improved if it contains gravel or sand whose grains are not too small and waterworn. The dark colored, more or less sandy alluvial clay of the lower Mississippi valley, locally known as "buckshot," is the most advantageous soil there obtainable for the purpose; but unfortunately its occurrence is by no means general. Sand, or more often sandy mixtures, are more frequently found; and such material is the less effective, and therefore requires the greater width of levee, in proportion to the preponderance of the sand, particularly if its particles are small and rounded. Even the lighter earths, such as loam or peat, are often preferable to the latter kind of sand deposits. Evidently the cross-section of a levee must be the greater where the earth available is the lighter, less cohesive or less impervious.

Consideration of the infinite variability of characteristics of alluvial

soils leads to the conclusion that it is often possible to reduce the permeability and increase the stability of embankments by a certain amount of control of the methods of construction. However, the requirements of economy rarely allow any considerable separation of the better materials from the poorer, for the purpose of controlling their advantageous placing in the levee; although some Holland dikes have a layer of clay on the river face as an impermeable covering of the core of sand, and the methods of construction used upon many levees of the Sacramento valley have permitted the entire encasement of the body of sand by a firmer earth. Those measures that do not materially increase the expense of construction will often permit some reduction in cross-section.

Probably the best criterion available for judging of the sufficiency of a levee is the slope of its line of saturation. The steeper inclinations are characteristic of the less permeable or better drained embankments; and because their stability is not secure when that slope of saturation emerges above the base on the landward side, it is evident that the width of a homogeneous levee may be reduced by all available methods of construction or choice of materials that will have the effect of increasing that slope, unless the foundation is defective. Its minimum average inclination in the levees of the Mississippi is usually between 1 on 4 and 1 on 6; but it fluctuates much, especially on account of the varying quality of the earth composing them and its manner of placing, and also with the duration of the high water period. The latter factor sometimes of itself makes necessary a broader section than would otherwise be required; as on the same river, where the duration of stages reaching above the natural banks often continues from fifty to a hundred days and sometimes amounts to 50 percent more, although the period of full exposure rarely exceeds two months. The actual mean surface of saturation within the levee is curved in transverse section, with its river end at the surface of the water where its slope is quite steep, the inclination gradually reducing as it is followed into the embankment until it approaches horizontality in the vicinity of the inner toe.[1] As the high water is prolonged the radii of curvature lengthen and its position consequently rises, thus saturating a greater portion of the levee and gradually reducing its stability.

A horizontal width of crown of several feet is necessary, both to preserve the crest from a progressive degradation and to allow suf-

[1] Transactions, American Society of Civil Engineers, Vol. 39, p. 236; "Standard Levee Sections" by H. St. L. Coppée.

ficient width for patrol and emergency work at times of flood. Any excess width at the crown, however, not only requires an added volume of earth that is not there needed for stability, but actually augments the weight of the surcharge above the slope of saturation; and so may materially increase the danger of sliding of the upper part.

The required width of base of a levee is chiefly secured by giving its sides the necessary inclination, which is the flatter as the levees are the higher, in addition to the need of thus compensating for the imperfections of the materials, workmanship, or foundation conditions. The declivity must, of course, never exceed the slope of repose of the soil under the circumstances attending its use. A very general practice, in the construction of the higher levees, is to enlarge their base by a bank of earth on the side away from the river. This banquette is planned to join the inner slope of the levee at a level

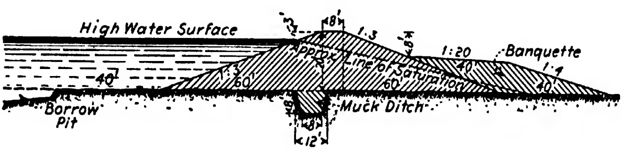


FIG. 90.—A typical levee section.

safely above the surface of saturation; its top inclines enough to allow a natural drainage and has a width dependent upon the nature of the existing conditions; and its side slope is usually somewhat flatter than that of the levee above it, as indicated in Fig. 90,[1] showing a standard levee section of the lower Mississippi River, 20 ft. in height. The particular service of the banquette seems to be a fourfold one; it acts as a buttress to support the upper part from sliding, it furnishes a massive spread of material which offers the most efficient method of preventing leakage underneath the embankment, it most effectively strengthens a weak foundation against lateral displacement and so serves as a counterweight against settlement of the levee, and it lends itself to a most convenient enlargement whenever this is found necessary, especially at times of danger during the continuance of high water. The last reason is especially pertinent, in connection with the need of economy in construction with a material of variable and uncertain behavior, because the

[1] Brochure 4, Fifth Question, First Section, of the Eleventh International Congress of Navigation.

side slopes may be correspondingly steeper when the banquette is employed and the volume of earth is placed where it is particularly effective.

In the consideration of a levee built of materials of such irregular composition and qualities upon foundations varying so much in character, where the methods of construction that are available for securing increased efficiency are limited, in order that the distribution of material as governed by the form and dimensions of cross-section will combine a maximum of effectiveness with a minimum of cost, it is very evident that adequate design requires a particularly discriminating comprehension of all the essential features involved; so that each detail may be definitely adapted to the special conditions encountered.

When noting characteristic dimensions of some of the important levees of the world, it is well to remember that they refer to the normal or usual proportions of each; being modified, as above discussed, whenever local conditions require a departure from the typical values given. It is a very general practice to use steeper slopes, or to omit the banquette, on embankments of moderate height; under favorable conditions both expedients are allowable. In Europe they frequently have a considerably greater width at the top, mainly because of their frequent use for roadways; but this is often accompanied by steeper side slopes. Experience has been the chief factor in arriving at the dimensions employed.

The great levees of the lower Mississippi ordinarily have a crown from 6 to 10 ft. in width, and side slopes of one on three. The top of the banquette is about 8 ft. lower, and it has a landward slope of one on twenty to one on ten; its width varies from 20 to 40 ft. and its side is built to an inclination of about one on four, as illustrated in the preceding figure. Not a few of the embankments of the lower basins omit the banquette; and, instead, increase their inner slope to about one on four, and sometimes to one on six for the higher ones. The levees of the upper Mississippi and the tributaries, being smaller and generally of a better material and less severely taxed, have usually a crown width of 5 or 6 ft., a river slope of about one on three and a landward slope of one on two or two on five; with no banquette except in special cases, such as at the crossing of deep affluents.

The larger river dikes of Holland frequently are from 12 to 25 ft. wide on top, the inclination toward the river ranging from one on two to one on three; and at the opposite side, from two on three to

two on five. Where banquettes are employed their crest is placed about 8 ft. below that of the levee, they are from 10 to 40 ft. in width, and their side slope varies from one on two to one on four; more often they are omitted, the landward slope being correspondingly flattened.

Those of the German rivers are normally given a crown width of 6 to 16 ft.; their inclination on the river side varies from one on two or three for the smaller, to one on three to five for the larger and more exposed defenses; that on the inner side is generally one on two or two on three. The occasional banquettes are from 6 to 10 ft. lower, have a top width of 6 to 16 ft., and a side slope of about one on two. Some of the Russian levees are quite similar in cross-section.

The more massive embankments of the Po have a crest that is from 16 to 30 ft. wide, with the inclination on both sides averaging one on two. Banquettes are quite common, rising to within 5 to 10 ft. of the elevation of the levee crown, the higher ones being employed in the more exposed parts; their width similarly ranges from 15 to 30 ft., and their side slope is also approximately one on two. In those localities where their assured integrity is of the utmost importance, there are two banquettes of equal width; the higher being at an elevation intermediate between that of the lower banquette and that of the crest of the embankment. The Austrian levee sections resemble those of northern Italy, except that the slope on the river side is more often one on three, and the banquette crest averages higher. The largest embankments on the Tisza River have a volume of about 41 cu. yd. per foot of length; on the Loire, 42; and on the Po, 46. One of standard form, 20 ft. high on the lower Mississippi contains 50 without, or 68 cu. yd. per foot with a banquette 40 ft. wide.

84. Their Construction and Cost.—It is the general practice to completely clear the proposed site of all trees, roots, brush and other vegetation, leaf litter, light mold, etc.; and then to plow the surface of the ground to a depth of 6 or 8 in. in the endeavor to secure a thorough union between the embankment and the earth on which it rests; but when it is built upon an unstable soil, the sod is left undisturbed in Holland. It is then customary in this country to construct a muck ditch extending longitudinally 6 or 8 ft. nearer the river than the crown of the levee will be, as indicated in Fig. 90 (page 343). Its purpose is partly to explore the foundation, securing a more definite knowledge of the qualities of the local soil and assuring the removal of buried logs, roots, and other causes of weakness;

and partly to insure a more definite bonding of the old to the new and diminish the danger of leakage by filling the trench with the best material obtainable at the locality, thoroughly tamped into place. Breaks have occurred from lack of their sufficient employment; and instances have been reported where their adoption has signalized the success of levees when similar ones in the same locality, without them, exhibited a dangerous amount of seepage. On the lower Mississippi they are frequently 8 ft. deep, 12 ft. wide at the top and 8 ft. wide at the bottom; although there, as well as in other parts of the country, their dimensions often do not exceed half those given where there exists a superior quality of earth. On the contrary in cases of especial importance, muck ditches have occasionally been made 50 percent larger; and in Holland, where they are sometimes employed, they are filled with clay puddle extending downward to clay substratum when its depth is not too great. Unfortunately a firm, impervious foundation is generally beyond reach on the lower Mississippi. Test pits are resorted to if the actual character of the soil requires further examination.

The earth for a levee is taken from shallow borrow pits on the river side of its location. Experience has led the Mississippi River Commission to specify a minimum distance of 40 ft. between the toe of the embankment and the nearest edge of the borrow pit in order to minimize the danger of erosion; and to limit its side slope to one on two, and its depth to 3 ft. at this edge and to 6 ft. at the side next the river, with a gradual slope between, for the additional purpose of reducing to a small amount the danger of weakening the levee foundation or of increasing the seepage underneath. On smaller rivers and under circumstances of less severity, the requirements are not so extensive; as on the upper Mississippi, where the minimum width of berm is usually 20 ft.; in Holland the corresponding distance is 33 ft. In order to further discourage erosion, resulting from the excavation acting as a convenient channel for a parallel river current that might endanger the levee by its rapid enlargement at times of high water, it is customary to interrupt its continuity by prohibiting the disturbance of the batture at intervals. The traverses, thus left, separate the borrow pits quite definitely, and sometimes even induce a deposit of silt that gradually fills them. Meanwhile the depressions are drained by cutting a ditch from their lowest edge outward. The prescribed interval between traverses must not exceeed 300 ft. on the lower Mississippi, their top width is at least 10 ft. and they have side slopes not steeper than one on two;

in Holland their minimum top width is 20 ft., and their maximum spacing is 330 ft. Under circumstances of particular danger from the excavated area, unusual precautions are sometimes taken; as a checker-board arrangement of smaller borrow pits on the lower Colorado river, where the alluvium is extremely erodible. Excavation on the land side of levees is generally considered a menace to their stability because of the resulting increased danger of seepage underneath; instances have occurred in which this has undoubtedly contributed to their destruction. In case the borrow pits must be located inside the embankment line, the width of berm is doubled or tripled; its specified minimum is 100 ft. on the lower Mississippi, and 60 ft. on the upper river.

The excavated material is freed from frozen lumps, roots and all foreign matter, and is deposited on the embankment in layers. In Europe a large part of the earthwork is moved by wheelbarrows or horses and carts, is often placed in courses about a foot thick sloping away from the river, and then is compacted by tamping or by the controlled passing of the teams. In this country the necessary restrictions imposed in regard to width of berm and the depth and limited length of borrow pits have resulted in the very general use of wheel scrapers and teams, depositing the earth in horizontal layers not exceeding 2 ft. in thickness. Drag scrapers (scoops or slips) and Fresno scrapers are considerably employed in different localities; and wheelbarrows are used on work of limited extent, reducing the courses to a depth of 1 ft. or less and ramming the earth in place. The berm must be preserved, uninjured. Elevating graders and wagons, cableways, etc., have been sometimes employed, and occasionally steam shovels with locomotives and dump cars on the heavier work; but the necessary conditions which limit freedom of excavation, and the extended character of the work, have not proved particularly favorable for the use of the more elaborate machinery upon the levees of most navigable rivers. The comparatively dry earth composing the embankments, although considerably compacted, reduces in volume by gradual settlement and consolidation. The percentage allowed for shrinkage in Holland varies from 5 to 15; and in the United States, from 10 to 15 percent when scrapers and teams are employed, and 20 to 25 percent on wheelbarrow construction. In India it is stated that the nature of the soil and the operation of building by using bullocks and scoops are so effective that no shrinkage allowance is necessary.

Occasionally the banks of navigable rivers are so low and so little

subject to erosive action that levees can be built upon them as spoil from dredges working in the stream. Much construction of this kind, along the lowlands of the Sacramento River, has been done by clamshell dredges with buckets having a capacity of about 5 cu. yd.; but the sandy silt from the river bottom, placed near the bank, has often been less stable and resistant than was found desirable. A recent levee of the Natomas Consolidated District, averaging 15 ft. in height, was planned to minimize these objections by locating it 150 ft. from the edge of the river and protecting the body of the embankment by a crown and sides of a firmer earth. Drag line excavators, having booms 100 ft. in length, built side dikes with the material drawn laterally from a rather wide and deep cut-off trench extending along the center line of the proposed levee. Into this was carried the spoil from a hydraulic dredge excavating in the river channel, the discharge pipe being carried to the site upon trestle bents. After the sand fill had reached the necessary elevation it was covered by a layer of earth drawn up from the excess height of the side dikes previously placed. The sand core is the most inexpensive material for the body of the levee, and it also prevents the somewhat dangerous operations of burrowing animals. The cost is stated to be about 8 cents per cubic yard.[1] Such levees, made from dredged material, are often given a considerably broader crest than those built by dry excavation.

The stability of levees is frequently increased by reducing the extent of their saturation by drainage. A ditch properly located along the inner foot of the embankment is often of much service in preventing the danger of its failure by sloughing. Occasionally tile drains have been placed beneath the inside edges, with excellent results. The opportunity, thus available, of increasing the stability of levees at times of greatest stress and least resistance, is an auxiliary advantage of real importance.

The employment of an impervious diaphragm, extending from top to bottom of the embankment or below, and situated as far from the inner side as is practicable, has often been proposed. This is intended not only to prevent the saturation of the material inside its position, and thus to make the stability of the levee so much greater that its dimensions might be reduced; but also to provide a barrier against the occurrence of continuous fissures or other channels in or under the levee, through which streams of water would flow at flood times, appearing at the inside in the form of "sand

[1] Engineering-Contracting, Vol. 41, pp. 492–93.

boils" which sometimes are fatal to its permanence. For this purpose vertical core walls of clay puddle or concrete, sheet piling, or a concrete revetment on the river slope have occasionally been employed; but in most circumstances the theoretical advantages are overbalanced by practical difficulties. It is rare that clay of the necessary quality is obtainable in the locality; and, even if it were such causes of weakness as the work of burrowing animals would not be prevented. If a concrete core is contemplated, its use must be confined to those places where the bearing power of the soil is sufficient to carry the greater weight; and that this restriction is often serious may be judged by recalling the rather numerous instances in which settlement occurs even under the much lighter load of earth. Further, either kind of core wall would be only partially effective unless it should extend to an impervious stratum beneath the levee; and this is frequently beyond reach, especially in the valley of the lower Mississippi River. Sheet piling has been occasionally employed to cut off the underground flow; but this becomes useless through decay, unless it is continually saturated; and, when 20 ft. long, sheet piling adds $3 or $4 to the cost of the levee per linear foot. Besides these limitations there are the objections arising from the increased expense, not only of the core walls themselves, but also because of the reduced freedom of operations in placing the earthwork in the embankments. The result of all these considerations is the almost universal practice of designing levees as simple earth embankments of sufficient cross-section. The most promising method of excluding water apparently is the employment of a concrete revetment upon the outer slope.

The expenditure for the construction of levees by the federal government, during the past thirty years, has averaged about 23 cents per cubic yard of volume after shrinkage is allowed for. Contract prices differ much with the locality and season, the extreme variation from the mean cost just given being about 50 percent. The average expense per mile of levee upon the lower Mississippi has probably exceeded $40,000. On the contrary, the first cost of similar work in Hungary, under conditions there prevailing a half-century ago, was not much more than one-third as great.

85. The Maintenance and Repair of Levees.—To fortify the surface of levees against the denuding effects of time and weather and to more effectively defend them against surface disintegration during periods of high water, it is customary to encourage a vigorous growth of grass over all the surface which has previously been trimmed

to regular slopes. On the lower Mississippi tufts of Bermuda grass, planted at intervals of about 2 ft., rapidly spread and soon form the very desirable dense sod. Blue grass and varieties of fescue are more effective in more northern climates. In some European countries it is customary to remove the turf from the site of a proposed levee and preserve it for sodding the more important parts of the finished slopes; the remainder of the surface is then sown with clover and grass. In this country and in Holland, the growth of brush upon the embankments is discouraged because it is believed to lessen the vigor of the development of the grass and to tend to loosen the surface soil; but this practice is by no means universal. For example, willows and similar growths are often planted on the side facing the river, and all herbage is encouraged in India because it is thought to most thoroughly bind the earth, and to furnish the most effective natural defense against the currents and waves at times of flood. Trees are everywhere kept from growing on or near the levees because their roots loosen the soil; and, when they decay, channels are left into which the water penetrates, often to a dangerous extent. On the lower Mississippi, they are cut for a distance of a hundred feet from the embankment, at both sides. Trees are, however, an advantage when at some distance from the levee because they very much reduce the eroding force of waves and currents at high water.

In places of especial exposure it is desirable to revet the outer slope, using methods similar to those decribed in Articles 66 and 67. When the width of levee permits, it is often less expensive to allow the embankment to form its natural flat slope of perhaps one on eight, under the action of the waves, than it is to incur the expense of revetment. When available, a covering of a cementitious gravel sometimes forms an excellent defense to the integrity of a levee; as in the valley of the Colorado River, where it was given a thickness of 15 in.

A proper maintenance also requires a periodical repair of places on the crown or slopes that have deteriorated; especially the filling of depressions and the choking of cracks or holes, particularly on the outer side. In case the enlargement of a levee becomes necessary the material should be added to the river side after the ground and exposed slope have been prepared as described for original construction. It has frequently been found that old levees, which were seriously leaky, have been made comparatively impermeable by constructing a muck ditch along the outer toe, which is covered by the new material of the enlarged section.

Although roadways are frequently found upon levees in Europe, they are generally discouraged in this country because of the more rapid deterioration resulting; when necessary, they are usually located on the banquette, preferably along the middle portion. Where roads cross an embankment, the ramp is ordinarily only wide enough for a single track, and is given a grade not exceeding 15 percent; but in Holland it is usually not more than half as much. Their side slope is made as steep as the earth will hold.

The destruction of levees by undermining is characteristic of rivers having eroding banks. The loss from this cause is particularly severe on the lower Mississippi, where it averaged more than 2,770,000 cu. yd. annually for the period 1900–1913, exceeding 57 percent of the amount constructed by the federal government in this region during that time. The definite remedy is the protection of the caving banks of the stream. This is an expensive procedure, but one which is being increasingly employed in places of especial importance, as discussed in Chapter VII. The fact that revetment is a very important agency in improving the navigability of this river, in addition to its service in preserving the levees from undermining, amply justifies a very much more extensive recourse to these works of defense.

The reduced velocities of the shallower part of the flood section next the levees, and especially the retarding effect of the trees and brush upon the batture, have the general tendency to here produce a deposit of some of the silt with which the stream is very heavily charged at times of high water. So advantageous is this, in decreasing the porosity of the soil, that it is often encouraged; as in Holland, where low brush structures, weighted with stone or brick, are constructed to more definitely promote the sedimentation that has thus raised the foreshore several feet above its original elevation.

Before the river banks are overflowed by an approaching flood, the levees are cleared of matted vegetation and are closely mown so that defects may be detected and precautions taken to put the embankments in a state to most effectively resist the flood waters. If, through any mischance, their thorough repair has not been annually maintained in order to secure a maximum resistance to high water conditions, a scrupulous filling of all cavities, fissures and other imperfections of the river slope contributes much to the efficiency of the defense; and surfaces of extended porosity, either apparent or known from previous experiences, may well be blanketed by as tenacious a material as can be procured, thoroughly tamped

into place. The ditches and other drainage facilities on the inner side should also be made effective. As the rising river reaches the levees, it is customary to arrange for the assemblage of the tools, apparatus and materials that may be needed hurriedly for emergency work, into supply depots located at intervals of a few miles. A patrol is also established, each inspector supervising not more than 20 or 25 miles of embankment so that no part of it shall escape his attention at least once a day. When the stage of the stream reaches the point at which any indications of weakness begin to appear, a sufficient force of men is assembled, under competent superintendence, to be ready to proceed with the needed defensive operations; and watchmen are employed in constant scrutiny for the double purpose of detecting any sign of defects as soon as they may become evident, and of preventing the malicious cutting of a levee to relieve the strain elsewhere. In Hungary, the watchmen cover an average distance of 3 or 4 miles, each; and on the lower Mississippi, each one patrols from 2 to 5 miles, the length depending upon existing conditions; places of developing weakness require the attention of workmen also.

Usually the first unfavorable appearance is that of small streams of water issuing at the base or from the surface of the ground inside the levee, or else from the lower part of the embankment itself. Experience has shown the fact that these are not generally dangerous as long as the water flows clear; but a muddy appearance indicates an enlarging channel which may very quickly develop into a breach, especially if the vein is several inches in diameter, or is smaller but traverses a quite sandy or loamy earth. Often such channels, in a clayey soil, gradually become choked naturally; and sometimes this action may be materially assisted by dumping earth on the outside of the levee where the river current is not too strong, if the outer end can be located. Occasionally sheet piles can be driven on the river slope to intercept the underground stream; but this procedure, like all others connected with emergency operations at such times, must be subject to experienced supervision to avoid the danger of aggravating the difficulty instead of terminating it. Other devices at the outside have occasionally proved successful; such as various sorts of temporary cofferdam or bulkhead construction in case the place of entrance is located. However, the usual method of dealing with such flowing streams or sand boils is to surround their inner end by a subordinate embankment of some convenient material, situated not too close to the mouth of the one or

more channels, and raised to such an elevation that the depth of the ponded water will approximately balance the pressure of water outside, diminished by the friction head lost by a not excessive volume of flow and seepage in traversing the passages. The crest is terminated at an elevation not less than 2 ft. below the surface of the river; otherwise practically all advantage is lost. If on or near the inner slope, the enclosing bank often is given a horseshoe

FIG. 91.—Hooping a small sand boil.

shape, as in Fig. 91,[1] which was built up by the very convenient but expensive method of using sacks filled with earth, which are most effective when fresh earthwork is concerned. If required farther from the levee, the site is completely hooped. The permanent remedy for such places of persistent leakage is often the covering of them by banquettes or by the construction of permanent sub-levees which allow the continued application of this principle of a counterhead of water; but a better plan generally consists

[1] This, and the three following figures, are from Report of Chief of Engineers, U. S. A., 1912, p. 3724.

in adding a thick blanket of a clayey soil to the outside slope, properly prepared to receive it, if it is a defectiveness of the embankment that causes the trouble; or, if the ground below is at fault, a banquette properly located on the outside forms a more perfect relief.

A very disturbing manifestation of weakness often occurs after the flood waters have stood for a long time or to a considerable height against a levee. It is the sloughing of the inner slope in large masses, due to the weakening of the soil by saturation. The

FIG. 92.—Holding a saturated levee.

danger of such slides can be diminished by preventing the inflow of water from the river side by methods above suggested, whenever that may be practicable; or, always, by carefully facilitating the drainage of the inner slope as soon as the exuding seepage appears in order to reduce the saturation to a minimum. The sloughing of embankments, beginning as it does at the lower third or half of the slope, progresses so deliberately that it is not considered an alarming event if proper attention can be given in time. The effective remedy is to cover the surface with a layer of brush, poles or small trees with only enough earth in sacks (or spread over the brush in a way to keep it from preventing a free drainage of the saturated slope beneath) to hold the brush in place. A view of such work is shown

in the immediate foreground of Fig. 92. Stakes driven into the slope, through the brush covering, and braced if necessary, will often so much aid in holding it in position that a considerably lighter weight upon it will be sufficient. The preservation of an unobstructed drainage of the saturated surface, underneath and through the brush, is essential; and any unnecessary weight upon it is a source of danger in adding to the tendency toward further sliding. If sloughing is not stopped it may progress until the crown of the levee is going, and a crevasse is then imminent.

Whenever the wave wash of severe storms threatens to cut through an embankment, recourse is had to some sort of temporary revetment. In Holland, such devices as weighted tarpaulins, fascines, or timber construction are frequently used. The latter is also much employed on the lower Mississippi, often in the form of a facing of inch boards nailed to stout stakes driven firmly into the ground at frequent intervals, inclining at about the steep slope of the crumbling bank, with their tops braced and anchored. The board facing extends downward to firm embedment in the levee, and to a height sufficient to break the force of the waves. More often the greater availability and effectiveness of sand bags leads to their employment for this purpose; and in places of particular exposure they are placed with their ends toward the river. The material for filling the sacks should, as much as possible, be taken from the projecting irregularities of the washing slope. In case an improper location or other unfortunate condition results in excessive scour by high water currents, a resort to mattresses, or even to deflecting timber cribs, has occasionally been necessary to save the levee.

When the overtopping of an embankment of deficient height is threatened, teams and scrapers are engaged in raising the crest with earth taken from the front, when time allows this to be done. It is more often the case that haste necessitates less extensive construction, such as a timber bulkheading raised upon the levee crest, similar to that just described but always firmly backed by a tamped ridge of earth, as illustrated in Fig. 93 (p.356); or a temporary barricade of sand bags, which is most used because it is particularly available and effective.

Emergency operations such as these, that are mainly due to present deficiencies of levees, require great expenditures that will be largely avoided when the required dimensions shall be attained; but which have recurred whenever unusual flood heights are reached. In the record high waters of 1912 and 1913 of the lower Mississippi,

the total cost of the extraordinary flood defense measures, of the kind described in the last few paragraphs, amounted to nearly one and one-half millions of dollars for the two seasons; or about $2400 per mile of embankment that required special work of defense. Thousands of men and millions of sacks were employed, largely in raising the several hundred miles of crest to an additional height of 2 or 3 ft., and often more. At Lake Beulah a line of sand bags, hastily constructed to hold a gap, held until it was more than 20 ft. high.[1]

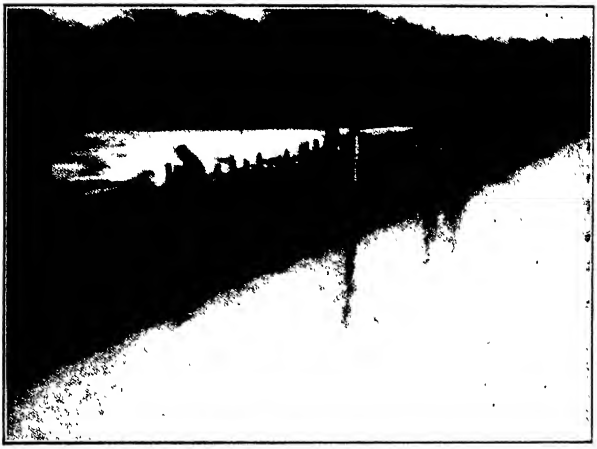


FIG. 93.—Raising a levee's crest.

The annual cost of maintenance of the levees of the Tisza, for a long period of years, averaged about 11 cents annually, per acre protected. The total yearly expenditure in Hungary, including pumping, administration, and all other operations of flood defense, is stated to average about 40 cents per acre, or somewhat more than 4 percent of the total cost.

Whenever the high waters break through an earth embankment it is an exceedingly difficult undertaking to close the crevasse, as may be judged from the tumultuous rush of the escaping flood illustrated in Fig. 94. The seriousness of the situation is the greater in the higher floods or when a tenacious sod upon the levee is lacking, and it becomes extreme when the soil is very sandy or

[1] Engineering News, Vol. 69, p. 803.

consists of a light loam. Not only is a closure frequently such an arduous and burdensome proposition that the attempt is foredoomed; but it is very expensive and, when successful, is often not accomplished until the stage of the river has fallen so much that a great part of the damage has been done. It has therefore been the usual custom, both abroad and in this country, to postpone the reconstruction at a crevasse until the flood waters have receded. Yet there have been instances of successful closure, especially on the lower

FIG. 94.—A crevasse.

300 miles of the Mississippi River where the soil is of a more tenacious character and the flood heights are not so great.

It is, however, always possible and wise to hold a crevasse from attaining an unnecessary width by protecting the rapidly caving levee ends, particularly because the volume of flood waters, pouring through to deluge the valley, is about proportional to the extent of the break. It is advisable to hasten such defensive work, because the rate of recession of each exposed end is many scores of feet per day. The method employed consists of the revetment of the exposed ends by temporary expedients, such as sand bags strung upon ropes and laid in successive lines until the whole slope is protected, one end being anchored and the other wrapping around the caving

ends; or by weighted sheets of tarpaulin or canvas, similarly placed; or by the employment of the more definitely effective construction of brush matresses sunk just above and made secure; or by spurs of timber cribs or rows of piles filled with brush and weighted usually with sacks of earth, built outward at a considerable angle from the uninjured part of the levee for a distance that will prevent its further cutting by the swirling currents.

The more usual method employed for closing crevasses is that of several rows of piles thoroughly braced, usually curving outward upon the batture from points safely back of the exposed ends of the levee, and filled with sand bags or weighted brush or fascines. The successful closure of the Hymelia crevasse in 1912 was finally effected by a thoroughly braced system of piles in which heavy sheet piling on the outer row was substituted for a solid fill.[1] Timber cribs are also sometimes used. Occasionally the site of a crevasse is accessible from a railway, and in such cases its equipment may be advantageously employed; as in the case of the final closing of the Beulah crevasse in 1913. A trestle was built above the line of escaping waters, upon which a temporary track was laid to allow the dumping of material from the cars.[2] After filling the width of 1148 ft. with about 48,000 cu. yd. of stone, the moderate flow through the rock fill was completely checked by the dumping of about 145,000 cu. yd. of earth upon its river slope.

The closing of great crevasses on the Mississippi River has cost from $30,000 to $100,000 each, or from about $50 to $100 per foot. Their occurrence will become very rare when the levees shall be completed, but even such large emergency expenditures are justified by the immense property values affected.

[1] Professional Memoirs, Vol. 5, pp. 57–77.

[2] Engineering Record, Vol. 68, pp. 4–6.

CHAPTER X

THE CONTROL OF THE CURRENT

86. The Value of Concordant Flow at Different Stages.—The undoubted influence of a directing control of the current of a river, at low water stages, has been noted in connection with the consideration of many of the natural phenomena and of types of works of improvement which have been the subject of examination of this volume. In the preceding chapter a similar relation existing between levees and channel conditions is shown to constitute the actual occasion for regarding them as at all an aid to navigation. This has left untouched the consideration of the influence of the banks and of artificial structures at intermediate levels upon the effectiveness of the navigable channel.

The low water margins of a river do, of course, constitute the dominant influence upon the character of its channel dimensions at that stage; but it would be most unfortunate to overlook a similar effect occurring at higher stages of the stream, although its importance reduces as the river rises. Navigable rivers usually carry flood volumes fifty, a hundred, and sometimes two hundred or more times as great as their low water discharges; with velocities doubled, tripled, or perhaps more than quadrupled; and accompanied by a rise of surface usually not less than 20, and sometimes exceeding 50 ft. The enormously increased eroding and transporting power of the currents at such times generally results in loading the stream with silt to such a degree that, when the velocity and volume become less as the river falls, it can no longer carry its burden and great quantities of sediment are deposited. Where the path of the high water axis of flow parallels that at low water, a situation exists which causes the accretions to form in places which do not directly affect navigation; and these favorable features are typically characteristic of conditions of flow in the curving portions of rivers. On the contrary the situation is quite unfavorable in straight parts, where the high water flow is also approximately straight but the course of the low water stream winds from one bank to the other; and it is especially adverse at the places of reversed curvature of the river bed,

where the low water channel passes from the concave bank at one side to the concave margin at the other while the high water stream has the opposite tendency at the crossings of inclining toward the convex bank. There results a considerable difference in direction of flow at such places, as typically idealized in Fig. 95. The immediate effect of this lack of coincidence in direction is a tendency for the low water channel to fill with sediment at the higher stages; and the ultimate result produced by the falling river is either the scouring out of the partly filled channel as the stream approaches the low water stage if the divergence of the two lines of flow is not too great, or the failure of that desirable eventuality in case the variance is so considerable that the potential serviceability of the river becomes unavailable for that reason. Probably it is not prac-

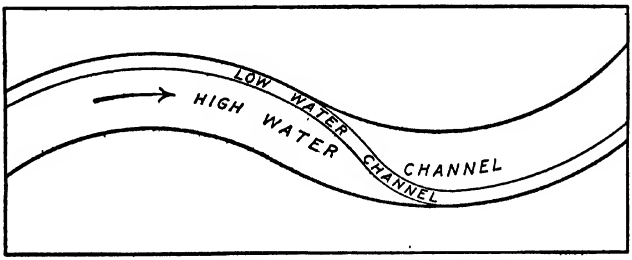


Fig. 95.—Nonconcurrent flow at different stages.

ticable to secure that entire concord in the flow of a river at all stages which is theoretically desirable; yet in those portions of a stream where its satisfactory regulation may be particularly difficult, it is well worth the endeavor to secure such added assistance by planning as close an approximation to the coincidence in position and direction of the axes of flow as may be practicable, especially at the medium and lower stages.

Besides this too often ignored question of the great advantage of approximately harmonizing the conditions of flow at different stages in order to aid in the preservation of the low water channel, there is the desirability of a proper coördination of the river sections accommodating the changing volumes of discharge in order to secure the avoidance of too great variations in the velocity of flow in the interest both of the efficiency of the improvements and of the river's safe and effective navigation. This latter question has received much attention in Europe, where the adjustment of the medium

and high water cross-sections to secure reasonable velocities is also carefully studied in connection with the necessity of coördinating the flow of the higher stages with that of the low water channel to prevent the deterioration of the latter.

It is therefore very desirable to consider interdependently, not only the low water channel section to which the greater part of this book is devoted; but that much larger section of the medium stage which generally consists of the natural river bed and banks unmodified, except by the revetting of the concave bends where necessary to prevent their caving, and so requires the careful adjustment to it of the position of the contracted low water channel in order to avoid the obliteration of the latter by sediment; and also that of the flood stages, which must be correspondingly more extensive in order to prevent too great velocities and excessive flood heights, as well as to assist in minimizing the objectionable deposit of sediment in the regulated channel, and which is particularly controlled by properly located levees.

This coördination of a river's channel sections at its several stages typically results in stepped terraces and in moderately sloping banks, planned as advantageously as conditions will allow with reference to the effects upon the low water channel. In low-lying valleys even the mean stage may require artificial banks; as in Holland, where low dykes are sometimes necessary to confine the river at medium stages to a width perhaps double that given to the improved low water channel, and flood dykes at still greater widths to restrain the highest waters.

87. The Significance of a Directing Control.—At different places in the preceding discussions there have appeared various indications of the particular import of a guiding control of the direction and energy of the current, in contradistinction to the problem of directly dimensioning the channel. This has appeared in connection with such considerations as that of sloping sills and similar structures which have been found to exhibit a salutary effect when employed under certain conditions of service; and that of some pioneer investigations to determine the form of groynes and sills which will not only offer the least disturbance to flow, but which will at the same time exert a beneficial control upon it. This molding influence has been shown in connection with the marked differences in channel conditions which have been found to result from variation in the shape and succession of the cross-sections provided for the low, medium and high water stages; and it has been particularly evident

in discussing the shape and sequence that should be given to the longitudinal curvatures of the river, and the preponderant influence of the concave bank (whether natural or artificial) upon the channel as the current flows along that margin in its onward course. The fact that effects of great moment result from the character given to works of regulation has also appeared in connection with the unsatisfactory consequences attending the construction of such works as bank heads and many spur dikes, their inability to produce regular and stable channel conditions resulting from the fact that they obtrude prominent obstacles at considerable intervals which enormously localize and concentrate the constraint, instead of applying the more reasonable principle of a continuously operating influence whose intensity at any place is therefore comparatively insignificant, and whose modifying effects are consequently gradual and regular, as practically accomplished in revetment.

The significance of a directing control of the current has also been evident in many other instances, such as the intimate relation existing between the channel conditions at any point and the exact character of the influences occurring for a considerable distance, especially of those above. The partial ineffectiveness of many works of regulation is often due largely to changing conditions upstream, which may produce great effects even though the cause is comparatively as obscure as the throwing of a railroad switch which carries the train to another region.

Not only will investigation and experience perfect the known methods of regulation, but new ones will undoubtedly be developed. The probability of such eventualities is not only indicated by the remarkable evolution characterizing operations for the improvement of the navigability of rivers up to the present time, but also by some recent developments. One of the latter, which may result in the assistance or even the displacement of the present standard method of widening a river channel that is too narrow but unnecessarily deep, by the use of sills, has been a subject of study by the Royal Experimenting Establishment for Hydraulic Engineering and Shipbuilding at Berlin. The experiments[1] on the models included inclinations of the concave bank varying from very steep ones to those which were quite flat. With slopes of 1 on 2 or less the channel was characteristically deep and narrow; but at less inclinations than these there was a considerable reduction in maximum depth and a marked movement of the deep water away from the concave

[1] Zeitschrift für Bauwesen, Jahrgang LVI, S. 337–338, and LVII, S. 70–71.

bank with an accompanying deepening all the way to the opposite margin. The advantage secured was most notable in passing from a slope of bank of 1 on 2 to 1 on 3. It is stated that the degree of improvement obtained by this procedure was found to be more marked than by the usual method of employing sills. The very considerable movement of the thalweg away from the concave bank produced by thus flattening its slope would not only favorably affect the navigability of the stream, but would very much decrease the severity of the action of the current upon the works of regulation built along that side.

The absolute responsiveness of river currents to the natural conditions of regimen, as well as to the character of the regulation works constructed to secure the improvement of its navigability, is unquestioned. Yet there is a wide difference of opinion in regard to such a question as the advantageous details of design for a curved portion of a river. The molding influence of the concave margin is generally recognized by all hydraulicians, including those who favor a complete contraction; but there are others who consider that a more economical and effective regulation is attainable by a correct design of the concave margin only, thus leaving the opposite side to adjust itself to the conditions so controlled.

88. Fundamental Character of a Guiding Influence.—Those who believe fundamentally in the training of the current realize the necessity of so designing the position and form of the directing (concave) side that the current is held from wandering from its designed path which forms the navigable channel. To effect this it is necessary to produce conditions in which the direction of flow of all the filaments in any truly transverse section approximates to the theoretical assumption of parallelism, or if not parallel they may exhibit a convergence in direction with beneficial results; but any divergence in a down-stream direction seems always to be accompanied by shoaling. Now two facts appear to be imperfectly realized; first, that the directions of filaments at a cross-section are often far from parallel, especially at places where the hydraulic axis crosses from the concave bank at one side to that at the other side of the channel; and, second, that a transverse section, usually considered normal to the bank line of the stream, is not necessarily normal to the mean direction of flow of the current as it should be in order to conform to hydraulic conceptions, but at crossings especially there is often considerable discrepancy between facts and the standard assumption. Of course the section should always be normal to the current.

In places where the natural or artificial guiding margin is diverging from the direction of the current, such margin is without influence; it must be parallel before it can begin to be effective, and even somewhat converge to the natural direction of flow in order to exert any notable influence on favorable channel effects. The location of training walls with reference to bank lines rather than by basing their position upon ascertained current directions has sometimes rendered them practically useless; or even detrimental. With

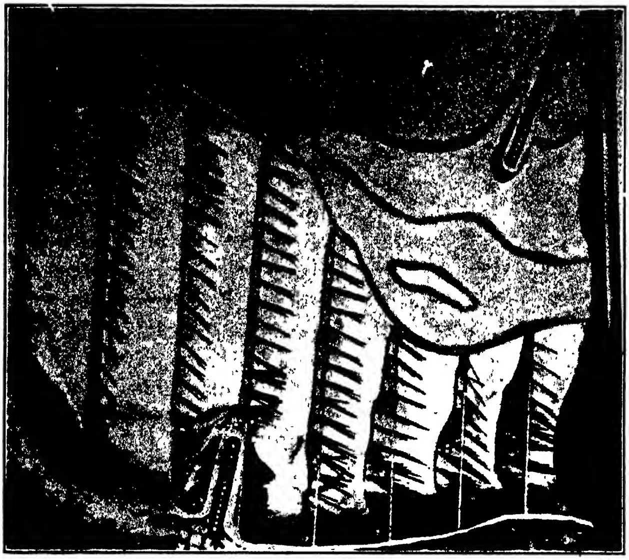


Fig. 96.—Deviation of filaments from an axial direction.

regard to the question of the parallelism of filaments, thousands of particular observations made in different portions of various Russian rivers about twenty years ago, while necessarily not delicate enough to detect conditions affecting oscillations and variations in direction in minute detail, yet showed many general facts which are significant. Considering these results as an integrated or summated effect of the numerous particles of water influencing the apparatus, it may be said in general that mathematical parallelism at any section of a river never exists. In the deep flowing pools the lack of parallelism rarely exceeds 5 degrees; but at shoal places, and especially at those alluvial deposits occurring where the hydraulic axis

passes from one bank to the other, horizontal differences of 20, 30 and 40 degrees were often found at a cross-section, and sometimes twice these amounts and more; while the vertical differences in direction were much smaller. In general, converging directions of flow of current are typical of pools and diverging ones of shallows. The preceding statements as summarized from observed conditions, illustrated in Fig. 96,[1] demonstrate the necessity of designing works of regulation only after local current conditions have been carefully determined; and when this information is then used in design it seems reasonable to expect that the results will be favorable to the extent to which the positive control of the current is thus achieved.

Of course the ultimate purpose is to secure the projected channel dimensions, but this may be more surely secured as an effect than as a cause. For, if the river carried no sediment and its banks and bed were non-erodible, as in a flume, the water would then necessarily occupy all of such channel to a definite depth, as indicated by hydraulic laws. But rivers are not without sediment, and a complete and continuous stability of bed and banks is financially impossible in most rivers; although cases have occurred in the regulation of streams where artificial works were so multiplied that the term "templates for the final shape" has been applied to them. These two facts, that a river is sediment bearing and that earth must always constitute nearly all the material of its channel perimeter must be fundamentally recognized in realizing the true significance of the reality that erosion necessarily results where velocities are excessive and deposits occur where they are deficient; and that these adverse effects are actually produced by local currents whose direction and velocity are far from conforming to those necessarily assumed in the usual theory of filamental flow, a fact especially likely to be encountered at just those places where regulation is most needed. With a consequent definite appreciation that these conditions are the actual origin of the impediments to navigation and that it is the irregularities of the current which constitute the undoubted cause of the difficulties which necessitate the improvement of rivers, it is apparent that the logical course is to directly attack the immediate agency of the local deterioration. It is therefore evident that a constructive control of the *vis viva*

[1] This and the two following figures are from Brochure 5 of the Sixth Communication, Section 1, of the Tenth International Association of Navigation Congresses.

of the river is the fundamental requirement of successful regulation, so that its energy may be discriminatingly directed to the advantageous molding of the channel, instead of either leaving it to its own eccentricities or expecting it to complaisantly conform to an arbitrary constraint which may be unsuited to its regimen, and so prove ineffectual.

89. Present Tendencies in the Regulation of Rivers.—Since all authorities practically agree upon the fact that the longitudinal channel form of natural streams should consist in an advantageous succession of curves, there is undoubted pertinence in the proposition (of those who insist upon always regarding the training of the current as fundamental) that the efforts of design should be concentrated upon making the concave margin a true director of the current so as to effectively hold the concentrated flow in a regular course. Those who apply the method of complete contraction agree in theory, but find that the laws expressing the relation between longitudinal curvature, velocity and volume of the stream, depth of channel, etc., are not yet formulated so that the design of the proposed directing margin can be definitely relied upon, leaving the opposite side to adjust itself naturally; and therefore they believe that the safe way is to plan the entire cross-section. The fact seems to be that, while there is urgent need for such laws to be experimentally determined, enough is now known to justify reliance upon a single controlling margin throughout the greater part of the course of the river, when carefully designed according to the best practice of the present; and consequently it is only such uncertain places as the crossings and those stretches whose form makes financially impracticable the employment of the advantageous curvature that may really require the rigid fixing of both margins. Many actual experiences corroborate this contention. One such notable example is offered by a portion of the Garonne which had been improved in the sixth decade of the last century by a continuous system of contraction works; but a score or more of years afterward the engineer in charge, who was for a long time a leading authority on the regulation of rivers, not only stated that they might have been omitted from the convex side except at the shoal places, but that the double line was not always necessary at the crossings, as an appropriate design of concave curvatures may accomplish the purpose.

In the present lack of definite knowledge concerning the laws of control of the river currents, the uncertainty of securing positive

results at places where the hydraulic axis passes from the concave margin at one bank to that of the other side leaves the standard method of regulation at such points a process of contraction. Against this method, which may in the future be referred to as one of expediency until the principles of directcurrent control shall have been developed, there is not only the theoretical objection that the procedure does not directly purpose to mold the active agency of channel-forming and channel-maintaining potency; but also the disadvantages that the second artificial margin is very expensive, that the currents within the contracted area are more or less irregular and therefore not fully effective, and that the complete contraction is harmful in unnecessarily reducing the channel area and otherwise restricting the flow at higher stages of the river.

However, when the attempt is made to express the principles of construction which should govern a successful project when improving a crossing by a directing control of the river current across it, instead of by contracting the channel, there is now only confusion and uncertainty. There are a few instances on French and other European rivers where success has resulted from the curvature of the directing margin reducing gradually from the adjoining pools until the radius becomes very great at the shallow crossing where the reversal of curvature occurs. On the contrary in some Russian rivers the curvature of the "director of filaments" has been designed quite differently, and again with successful results. In some cases on the Dnieper and Pripete it is stated that, on the failure of contraction works to secure the required depths, current control by one artificial margin was planned and constructed with entire success. The principle of design applied in the latter instances followed the practice advocated by the French cases just referred to in preserving the favorable curvature naturally developed by the river in the adjacent pools; but instead of flattening the curvature of the directing margin more and more as the shallow crossing is approached, this plan was to continue the favorable curvature and even to reduce the radius on approaching the crossing so as to secure an assured continuance of its controlling influence where the natural currents tend most to dissipate, and thus to make the control more definite and to shorten the length of the channel crossing. The adjustment of curvatures and of the position of the end of the upper directing margin with reference to that of the lower curve of the opposite bank is admittedly a very delicate one. It was found that the upper concave construction should extend onto the shoal at

such a curvature that it would surely slightly cut the direction of the natural currents, and to a distance into the crossing of from one-third to one-half the width of the river naturally existing at the place; and from this point to change to a convex curve as it is prolonged somewhat further down-stream in order to intercept the rebound of the current from the opposite concave revetted margin, ending when the current has thus been brought within the control of the latter directing bank.

The following illustrations are stated to typify and summarize a number of experiences encountered in the improvement of river crossings by regulation. Fig. 97 indicates general results obtained by contraction secured by the use of revetment, training walls and groynes as shown, the broken extension of the latter on the left side representing their later lengthening to attain a better channel at the right side than existed when they were shorter. Fig. 98 shows the alternative construction which was designed wholly on the principle of an advantageous guidance of the current. In both cases the concave banks must be thoroughly protected from erosion. In the former the numerous groynes on the convex part of the right bank divert an excessive amount of the flow to the left side and so unnecessarily deepen it; they also reduce the freedom of navigation at medium stages. In the latter figure care is necessary to so plan the directive training wall, which extends in a down-stream direction from the middle of the left bank, that its curvature shall be not much sharper than is surely sufficient to hold the current near its margin in order to avoid any unnecessary concentration, or velocities which would tend to make its navigability more difficult and would cause an increased erosive action where it strikes the right bank, and produce an excessive rebound and dissipation of the current from this lower concave bank. The careful adjustment of the length of this guiding margin was also essential both to secure a sufficient training of the current into the concave bank below and to avoid an excess of length which would accentuate the objectionable features just mentioned, because they are the more severe as the direction of flow at this place is the more nearly normal to the bank, and so make the control of the current more difficult in the upper part of the lower concave bank. The skillful adjustment of the convex margin opposite is also necessary in order to adequately restrain the rebound of the waters from the lower concave bank and so to deflect them as to bring them quickly within the guiding influence of the latter. The greater freedom

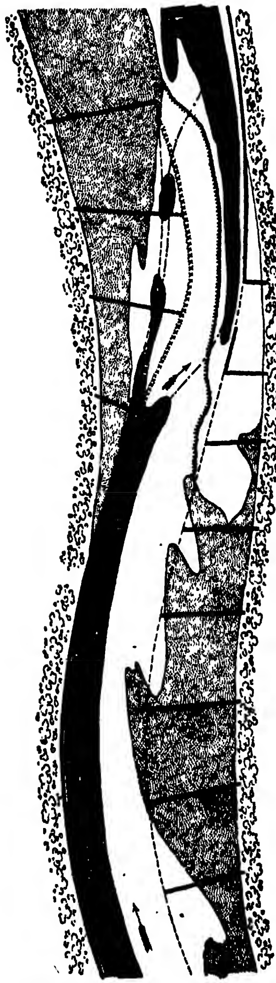


FIG. 97.—Regulation by channel contraction.

FIG. 98.—Regulation by current control.

of navigability, the less cost and the better channel secured by the second plan are all quite marked.

Thus, while the conception of regulation at the crossings by the direct control of its current is theoretically sound, there is as yet conflict in formulating principles to assure that result. Until experiment and experience shall determine such details many engineers will prefer to make sure of that degree of amelioration which is attainable by contraction at shoal places, even at much increased expense, rather than risk ineffective construction resulting from uncertainties in the application of principles which should control the fundamentally preferable alternative design.

It is this essential conception that primary attention should be directed to the guiding control of the river current's molding energy and direction, rather than immediately to the channel form, which marks a distinction in principle that will be significant of a real mastery. Even when works designed from this point of view may be not greatly dissimilar, yet the differences resulting from this correct conception will produce more effective results because the actual molding force is directly controlled, and thus will be averted even those lesser irregularities of channel which so often persist to partially defeat the purpose of the project.

As the knowledge of the regimen of rivers shall become more precise, and especially as their current effects both in natural and improved channels shall be interpreted on the fundamental basis of the complex action of flowing water under all conditions instead of primarily upon that of channel dimensions, there will surely result an increasingly definite series of principles for guidance in design with a correspondingly greater certainty of attainment of the expected results, and this will be accomplished at a less cost.

90. Theory Must be Adapted to Actual Conditions.—It is realized that the preceding discussion omits consideration of the practicability of adopting the exact design which is theoretically desirable. The wisdom of harmonizing abstract principles with conditions of service is undoubted. It is not good engineering if the plan fails to have a due regard for all the circumstances attending the project. Plans of operation must always be based upon existing conditions, with correct theory as the guide in securing a design which will successfully attain the purpose in view. Thus the abstractly most perfect proposition is often excessively expensive, and requires such modification as will make it feasible and yet effective. In no branch of civil engineering is this adjustment of theoretical principles

to conditions of construction and service more essential than in the regulation of rivers. The most desirable curvature and other elements affecting channel conditions are frequently unattainable because they would require modifications so extensive as to, really amount to a transformation of the river, rather than a practicable correction of pernicious details only. And yet it is worse than useless to permit the indefinite question of feasibility to eclipse or to so much change the theoretical requirements that the attainment of the intended results will be dubious from the start.

Examples illustrating this conflict of interests are almost as numerous as are the instances in which the training of rivers has been planned. The situation shown in Fig. 99[1] (p. 372) may seem to be an extreme one, but there are worse cases; as in places where a great width is occupied by a similar multiplicity of irregular channels, but with conditions so much less stable that the shoaler parts cannot remain fixed in position long enough to become islands. The suggested form and position for the improved navigable channel is indicated in the illustration, with the proposed works to accomplish this. The adaptation to local conditions is especially shown by the location of it through the sharply curving and deep "St. Aubert Bend" next the left bank, with a resulting necessity of a very slight curvature beyond. The concave margin had already been revetted from A' to B' in order to hold it from a further recession and increase of curvature, but the projecting point, from B' to B, was left unprotected until it should be eroded by the current back to the proposed line as shown.

Although there are other instances of the omission of revetment at portions of eroding banks, which projected beyond the position of the proposed fixed margin, until it should be sufficiently worn away by the currents, there are many who consider such adjustments impracticable. For example, a special board reporting on the resumption of operations on the lower Missouri River states that:

> "Caving banks must first be revetted, and in general they must be held and protected with the alignment found to exist when the work in its systematic progress reaches them. Frequently this alignment may not have the most desirable curvature, but this drawback must be accepted for the reason that ordinarily, within proper limits of cost, correction of the curvature by shaping, cutting or dredging the banks will be impossible. To permit a bank of unsatisfactory curvature to remain unrevetted until

[1] Report, Chief of Engineers, U. S. A., 1895, p. 4048.

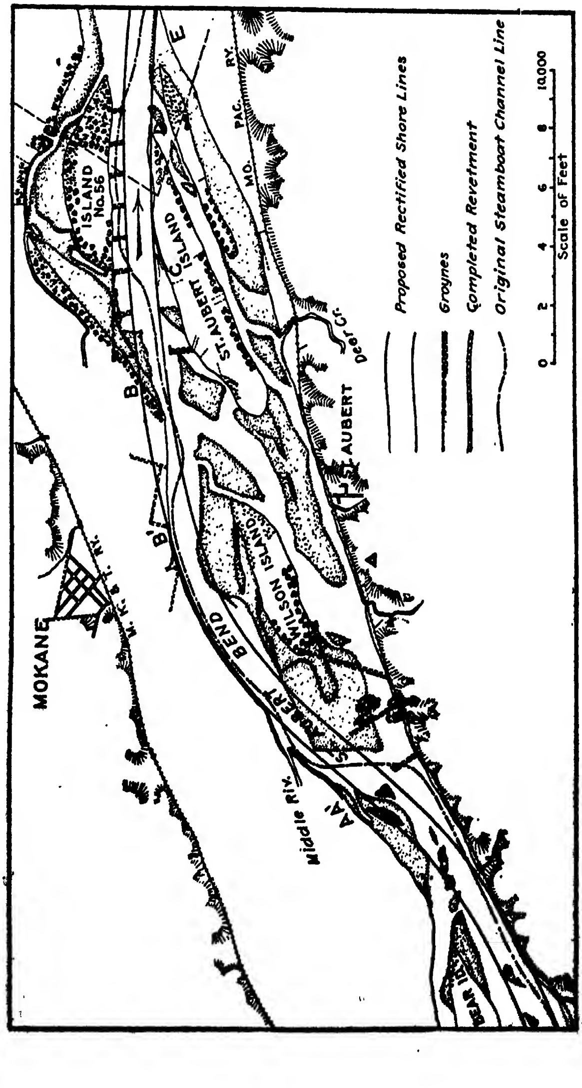


FIG. 99.—Locating an improved channel.

the river itself has corrected this condition is usually inadvisable. It will rarely be possible, no matter what precautions are taken, to arrest the caving and to protect the bank at the precise moment when the curvature has become satisfactory."[1]

Yet in reporting upon the most extensive and radical improvement of the lower Mississippi River yet proposed, although the volume of flow is much greater and the stream is therefore less dependent upon correctness of plan, a quite different opinion is held with regard to this question. It is believed that the generally preferable procedure, and certainly the truer principle, is that indicated in the following quotation:

"Bank protection . . . is to be placed at every alluvial concave bank when that bank, whether natural or artificial, shall have reached the proposed limit line. Some banks will be permitted to wear away considerably, and in some cases erosion will be directed to produce the desired alignment."[2]

Notwithstanding the fact that there seems to be a less pressing necessity of scrupulous attention to the principle of a directing control of the current at crossings which are improved by contraction than exists in the case of concave banks which have only the one guiding margin, the general plan for the improvement of the lower Missouri River, just referred to, does distinctly recognize the fundamental import of guiding the flow in concluding that "To regulate bad crossings a fixed and advantageous direction must be given to the channel, and it must be held there. . . . It must be emphasized that this [the desired result] is to be obtained by giving the flow across the bar a suitable direction rather than by its contraction."

The essential significance of securing the correct outline for the guiding margin is probably less adequately realized than is the supplemental necessity of holding it fixed in position when obtained. This fact is illustrated by subsequent modifications of channel form occurring in the portion of the river shown in the last figure. Although the groynes extending from the right bank, just above St. Aubert Bend, were actually constructed somewhat further upstream in order to less sharply divert the axis of flow from the right to the left side (and the traverse to the head of Wilson Island was found unnecessary) nevertheless the fact that the currents were

[1] H. R. Document No. 1287, 61st Congress, 3d Session, p. 10.

[2] H. R. Document No. 50, 61st Congress, 1st Session, p. 81.

even then given a direction which caused them to impinge too strongly against the upper part of the revetted bank actually did produce a considerable tendency to rebound toward the convex margin opposite and below. It will also be noticed that the protected bank of St. Aubert Bend is not regular; when the revetment was laid it had receded so far, in the vicinity of Middle River, that an obtuse angle existed just below its mouth; and this condition also tended to divert the current away from the concave margin. Even the effect of the rather sharp curvature here was not sufficient to counterbalance the combined influences of the two conditions just mentioned, and secure the desired axial course. The result, a few years later, was a divided channel opposite the lower half of the revetted bank, the arm next the shore being hardly half as wide as the one which cut through the sand bar that had extended continuously from Wilson Island to the channel as planned. A further effect of the imperfect control of the current was the fact that the expected erosion of the projecting point $(B' - B)$ did not take place, its face having been thus relieved from attack.

The significance of the complete responsiveness of current activities to the influences they encounter may be more advantageously expressed in inverted form. Its import to the civil engineer lies directly in the correlated statement that the position of the axis of flow, and consequently the channel dimensions, are in complete accord with the controlling conditions naturally existing or artificially imposed in the course of the stream. The careful determination of the position and form of works of improvement by means of a most thorough study of the detailed characteristics of the stream for a considerable distance above the place, at different stages and especially at mean and low water, will very much affect the adequacy of their influence. It is equally important to make those characteristics unchangeable, for reasons which are evident from operations already described; for a river is as responsive to changes of its regimen as is the pointing of an equatorial to variations in its control. There is no doubt that such a searching scrutiny preceding the design will profoundly aid in the direct attainment of the desired results; and it is believed that the magnitude and expense of the necessary regulating works may thus be reduced, often to an unexpected degree below that which follows from only partial knowledge of conditions, while constantly striving to make the completed plan approximate in principle as closely as it may to the theoretical standard of excellence.

91. The Import of Practical Considerations Exemplified.—The predominance of practical considerations upon the methods adopted for the improvement of a river is most apparent in the case of the lower Mississippi. It is very evident that this should be true in the distance of almost 300 miles from the mouth of the Red River to the Head of the Passes at the Gulf of Mexico because, with the exception of one place near the upper end of this stretch (which averages 84 ft. deep), there is everywhere a channel depth of at least 30 ft. Consequently there is no occasion to improve this part of the river for navigation, nor will such need occur under any requirements of commerce which can be at all expected. The direct reason for the construction of those works that have been built is the protection of caving banks whose further recession would endanger or destroy property of very great value, as in the vicinity of New Orleans.

The situation is quite different in the 764 miles between the mouths of the Ohio and the Red Rivers, for at many places the low water channel is deficient even under the present project of a minimum depth of 9 ft.; although the average is 31 ft. from Cairo to Memphis, 37 ft. from the latter city to Vicksburg, and 48 ft. in the remaining portion. Nevertheless the experience of thirty years has proved the advantage of maintaining the required navigable channel by the temporary expedient of dredging at the places and times found necessary, rather than by permanent works of regulation.

The reasons why dredging has been definitely adopted as the distinctive method of improvement for this portion of the river are based upon its especial adaptability and economy, due to a combination of conditions which are quite unusual. The river is extremely large in dimensions and volume of flow, its velocities are high, its bed and banks are distinctly alluvial in character and are therefore easily eroded, and consequently it displays a general instability of bed which is unequalled in all the history of river improvement in the world. Its greatness and the marked changeableness of its channel combine to produce a severity of conditions of unparalleled magnitude. These characteristics would make the improvement of the river by regulation a procedure of great expense and of very gradual accomplishment.

A second consideration, of equal moment, is the fact that the navigable depth required by the present project is very moderate in comparison to that which is possible of attainment. The 9 ft.

channel now required in the lower Mississippi corresponds, for example, to a depth of perhaps 5 ft. on the Rhine; but the latter river has actually been improved by regulation to a depth of 10 ft. Otherwise expressed, the statement of the situation is that the requirements of the present project involve only the deepening of shallow places at widely separated localities at low stages; and this moderate degree of amelioration is accomplished without delay and at a minimum cost by the periodic employment of hydraulic dredges operating at the particular places requiring improvement.

With the adoption of dredging more than fifteen years ago as the advisable method of securing the navigable depth required, there was no particular occasion to consider the auxiliary efficacy of bank protection or levees upon the improvement of the channel except in occasional instances; as when erosion had proceeded so far as to threaten a cut-off which would cause so violent a change in the regimen that its navigability in all that vicinity would be much impaired. Consequently the building of levees has been mainly for the especial purpose of protecting property and life against flood overflows; and the construction of revetment has generally been actually planned for the same service, both directly when placed in the vicinity of cities and indirectly to prevent the destruction of levees. There is little evidence that either kind of improvement has been designed with regard to the training influence that it might have upon the river currents. Contracting works were built about thirty years ago at two places, Plum Point Reach and Lake Providence Reach; but the large expenditures involved in their construction and maintenance and the fact that the resulting improvement of the channel was only partial, largely because their widely scattering locations produced only individually local effects rather than the desirable complementary control, both combined to cause the abandonment of further work of that kind. Thus experience has developed the general advantage of dredging alone as the distinct method of securing the navigability of the lower Mississippi under the present 9-ft. project.

The situation will be quite different when commercial interests shall require a much greater depth of channel. Of course the plan of improvement then adopted must necessarily coördinate with the essential protection of the valley from the enormous losses caused by destructive floods and by the caving of the river banks. The great and rapidly increasing economic importance of the region naturally subject to overflow and its prospective development to unrealized

proportions when its defenses shall be secure, will make the perfection of the levee systems and of bank protection a fundamental part of the general plan of improvement.

It is now generally recognized that a complete system of bank protection will also greatly benefit the navigability of the river. This will be due partly to the resulting exclusion of the enormous masses of alluvium eroded from the caving banks, and partly to the holding of the river to a fixed position in the concave margins; the general results of both these influences have been discussed more at length in preceding chapters. In the case of the lower Mississippi the latter effect would have a greater potency than any other single procedure toward approximating to that stabilization of regimen which is considered necessary if a permanent amelioration is to be attained; and a marked change in channel conditions would follow its relief from the present burden of silt that averages more than a million cubic yards of detritus contributed annually per mile of river from Cairo to Donaldsonville (885 miles), which is largely responsible for its excessively active bar-forming proclivities. The extent of the deepening at the shoal places which would result from both of these essential improvements is uncertain, but there is no doubt that it would be considerable.

92. The Advantage of Thoroughly Coördinating Theoretical Principles and Practical Conditions.—The preceding paragraphs have outlined the very extensive and expensive, but practicable and adequate methods of improvement which will be required to protect from disastrous losses the great alluvial valley of the Mississippi and to assist in the increase of channel depth which will in time be essential to secure that economy of transportation which is a fundamental factor in attaining a full measure of commercial development. Thus far the practical considerations of the protection of property, health and lives, with such auxiliary advantages as the preservation of uninterrupted communication by land routes and of the regular activities of agricultural and business interests, have dominated the discussion. The actual conditions, broadly considered in their entirety, lead inevitably to this view. The consequent subordination of the question of water transportation must not be allowed to obscure its consideration unduly. Fortunately one of the two great agencies of securing that protection of the valley will greatly improve the navigability of the river, as has been stated in referring to the salutary effect of bank revetment upon the regimen of the stream. The interests of channel improvement should not,

however, be limited to even this considerable degree of serviceableness; but the theoretical principles of river regulation ought to be kept equally in view, so that the adopted plans will assure a maximum practicable training effect upon the currents. To secure this guiding influence of revetment, in connection with its protective service, requires only the definite planning of its location and outline in addition to the usual procedure which generally ignores the latter very important factors.

There are other considerations which unite with those already mentioned to distinguish revetment as the most important characteristic feature in the complete improvement of the lower Mississippi. The navigation of the river is much easier and safer when the channel is at one margin that is regular in vertical slope and in longitudinal outline at all stages. The possible alternative of the extensive employment of contraction works to deepen the channel is a questionable proposition because the magnitude of the river and the instability of its bed would produce a severity of attack which would be exceedingly difficult to resist; and consequently the recourse to this latter method may well be avoided as much as is possible. On the contrary, because of its very form, revetment is the least vulnerable of all the types of structures employed in regulation and its foundation support is direct and continuous. It offers no obstruction to flow; in fact it encourages a steady run-off, rather than hindering it in the least. It also has the unique advantage of presenting an uninterrupted, and therefore a definite regularizing influence upon the stream at all stages when correctly placed, and so makes available a continuity of control which is of much importance.

So great an undertaking as the permanent and radical improvement of the lower Mississippi River will require a long term of years for its accomplishment. Meanwhile the needed navigable depth will be maintained by dredging, and the protection of the valley will be afforded by bank defenses and by levees. It would notably decrease the expense and time necessary for the attainment of that ultimate object if the revetment, which shall in the meantime be constructed from year to year, might as far as is practicable be located in such positions that the banks so held shall finally form integral parts of that comprehensive design which will most advantageously affect the future channel dimensions.

Such an anticipation of ultimate developments would decidedly assist in their accomplishment with not the slightest interference with the present serviceability of the revetment. Furthermore,

these results would involve no excessive increase in first cost; because the salient parts of the banks, which project beyond the line of the desired margin, may await the time when the eroding action shall have sufficiently worn them away; and those parts which have already receded too far may be built out by inducing accretionary deposits in the manner discussed in previous chapters. Concurrent advantages of much importance would accrue during that period of persistent preparation, because of the deliberate opportunity so presented to attain a thorough knowledge of the regimen of all parts of the great lower river in its relation to the character of the training works employed.

The avoidance of excessive expenditures, whenever a great increase in depth of channel shall become necessary, will be in proportion to the opportunity given for a progressive adaptation of plans to local conditions, both as they naturally exist and as they will be modified as the works advance. The general method of practically executing that advantageous procedure would probably involve the administrative division of the river into individual reaches of considerable length, in each of which the permanent works of regulation would begin at the upper end; the extension down stream being made only when the effects of those completed parts on the channel had become fully manifest and favorably developed; and the added length being temporarily limited to the distance thus clearly indicated as having been brought to a condition of advantageous control. The completeness of the success attending such a marked increase in channel depth by regulation would therefore appear to depend particularly upon the thoroughness with which the guiding influence of permanent works upon the powerful currents of this great river may practically and systematically be made to serve the purpose of its effective control.

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